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WATER SUPPLY AND
WASTEWATER SYSTEMS

Final Report

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16. Abstract (Limit: 200 words)
Seismic phenomena are described and a procedure is given to ensure that water and sewage systems are functioning properly in the aftermath of an earthquake. Large displacements, ground failure, and seismic shaking of pipes are shown to be the three major causes of pipeline damage. The effects of different soil media in relation to seismic shaking and damage of pipeline failures are examined. Potential damage to water and sewage treatment facilities is explored and basic failure modes are viewed in relation to treatment plant facilities. The need for site planning and design strategies to decrease the seismic vulnerability of water and wastewater systems is emphasized. Criteria are given for determining seismic induced loadings on water supply and sewage facilities. Emergency response and planning for earthquakes are discussed. The report is intended to provide information to water and waste system managers, designers, and planners.

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CHAPTER I

INTRODUCTION

A. BACKGROUND

Earthquakes are one of the most violent forms of natural disasters experienced on earth. The energy released is capable of causing massive destruction to unprepared communities. In the U.S., most earthquake activity is located in the far Western states. However, there are many other areas that have experienced strong destructive seismic events, although at a lower frequency of occurrence.

Over the past few decades, our level of knowledge about the causes and effects of earthquakes has expanded considerably. Similarly, we have developed new and increasingly more effective means of providing protection for life and property. The great majority of emphasis in seismic protection has been placed on buildings, dams and bridges. It has become increasingly apparent, however, that there are other systems whose protection is necessary for public safety and health, reduction in destruction of property, and maintenance of life style. These systems, known collectively as lifelines, are generally characterized by large collection or distribution networks. They include water supply, sewage disposal, gas and electric utilities, gas and oil pipelines, transportation facilities, and communications.

This report is concerned with seismic protection of water and sewage systems. Experience in past earthquakes has shown that disruption of these services can result in significant secondary losses from earthquakes (e.g., fire, disease and loss of industrial productivity). Water and sewage systems can be roughly divided into two major subdivisions: pipeline networks (distribution of water or collection of sewage) and treatment facilities. Over the past ten years, an increasing amount of work has been done on pipeline analysis, mainly on a theoretical basis. However, very little effort has been expended toward strengthening the design of treatment facilities. In an effort to partially fill this void, this project places major emphasis on treatment facilities. While protection of pipelines is discussed, the reader will be referred to reports of other work for more detailed analyses.

B. BASIC APPROACH

The purpose of this project was to develop information and procedures that would be directly useful in providing seismic protection for water and sewage systems. The report is intended to be used by managers, planners and designers of water and waste utilities.

The information presented is based upon four general sources:

1. Reports of earthquake damage to water, sewage or related facilities (e.g., industrial plants)
2. Theoretical analyses of the response of related equipment to seismic motion.
3. Information developed both in the U.S. and other countries on general design procedures (including the nuclear power industry).
4. Current design approaches for non-lifeline systems, including existing and proposed codes.

A significant portion of this project involved the collection and interpretation of data from past earthquakes. This activity indicated that with only a few exceptions, relatively little attention has been paid to investigating damages to water and sewer systems as compared to conventional structures. This could be due to a variety of reasons:

1. Most earthquake reconnaissance teams are composed of geologists and structural engineers with little interest in water and waste systems.
2. Most "earthquake engineers" are unfamiliar with water and waste systems and, therefore, are not aware of the significance of various damage modes.
3. Typically, damage identification to pipeline systems may not be completed months after the earthquake. Also, the damage surveys are

usually performed by utility personnel with no incentive to report their findings to the earthquake engineering community.

4. In recent years, earthquake damage to water and sewer systems has not had dramatic results (i.e., highly publicized). For example, with the exception of the 1906 San Francisco earthquake (fires), no deaths in the U.S. have been attributed to water and sewer system damage. As will be seen in this report, water and sewer system damage can be quite extensive. The lack of recent instances of "dramatic results" may be more a matter of luck than anything else.

Two basic types of design information were developed in this project: design criteria and design considerations. At the present time, there are no seismic design codes which were specifically designed for water and sewage systems. A suggested code of design criteria has been developed for major structural elements in water and sewage treatment facilities, based on extrapolation of existing or proposed codes for buildings and other structures. This suggested code is presented as a starting point for the development of a design code acceptable to both the water and sewage system utilities and the structural design community. Hopefully, the suggested code will generate discussion and evaluation by the various professional groups involved, ultimately leading to a widely accepted code.

For other equipment, design considerations have been developed. These are not presented in a structural design format but rather as suggested techniques of good practice. They are based on lessons learned from past earthquake experience as well as techniques to protect water and waste systems for general emergency situations.

At the present time, the state-of-the-art does not provide for accurate predictions of the local occurrence and physical manifestations of earthquakes and the corresponding impacts on specific water and waste systems. Without a dependable means of local risk analysis, the designer and planner must develop alternate means of rationalizing different levels of seismic strengthening. On one hand, most water and waste systems in earthquake prone areas are probably underprotected. On the other hand, total protection of all equipment would be very expensive, even if technically feasible. A method is presented in this report that identifies critical equipment that should receive primary attention. This is done by first identifying the overall performance goals of the system. A functional analysis is then performed to identify those functions and subfunctions that are required to meet performance goals. For each subfunction, corresponding equipment is then identified. The equipment is rated according to its importance in meeting overall system goals. This procedure allows a systematic review of water and sewage treatment facility design. It provides the framework whereby the planner or designer can set goals tailored for a specific installation which, in turn, provide the required input to the detailed design procedure through the functional analysis procedure.

It does not appear technically or economically feasible to provide sufficient design protection to completely avoid earthquake damage to water and waste systems. Consequently, utilities should be prepared to rapidly and effectively respond after a damaging earthquake. Information on suggested approaches to emergency response and planning has been developed to assist utility managers and planners in formulating their own plant-specific plans.

As stated earlier, the focus of this project was on treatment facilities with lesser emphasis on pipeline networks, since several other projects deal in depth with this subject. This report is intended to be of value to all water and waste utilities in earthquake prone areas. There are many different types of systems employing a variety of treatment unit operations and equipment in various combinations. In the interest of brevity, the most widely used unit operations and equipment have been emphasized: conventional water treatment and secondary activated sludge sewage treatment. There are many other types of systems that could be considered, particularly in waste treatment: land disposal, trickling filtration, lime-phosphorus removal, sewage filtration, ion exchange, etc. However, these other systems are typically composed of unit operations whose structural features are very similar to those presented herein. For example, ion exchange and sewage filtration are very similar to water filtration from a structural point of view. Lime-phosphorus sewage treatment is composed of chemical addition (same as water treatment chemical addition), coagulation and settling (same as water treatment) and recalcination (same as sewage sludge incineration). Trickling filtration of sewage is structurally very similar to activated sludge aeration (open, in-ground or above-ground tanks). Land disposal of sewage typically requires conventional secondary treatment, a storage lagoon and an above ground piping system to apply the sewage on the land. Except for the above ground piping system, the unit operations are the same as those covered in this report. Typically, rotating spray rigs, identical to those used in agricultural applications, are used. They are currently designed with highly flexible joints to allow for undulations in the land. This feature, along with the ability to rapidly connect or replace damaged pipe and extensive on-site storage, provides a high degree of protection against lengthy system downtime.

C. REPORT ORGANIZATION

The intent of this report is to provide extensive, useful information to water and waste system managers, designers and planners. The report has been prepared in a "manual" format. However, it should be recognized that the information, recommendations and suggestions contained herein have not been formally adopted by a regulatory or professional group. It is hoped that this report will, however, provide the stimulus to that end.

Chapter II provides background information on the physical manifestations of earthquakes in general. It is very basic in nature; additional information, if desired, can be found in a number of earthquake engineering texts. Chapter III presents a discussion of performance goals and functional analysis. This approach is used to set the required performance levels for the design of equipment. Chapter IV offers a discussion of potential damages to water and waste systems, primarily based on past experience. System and site planning concepts are given in Chapter V. Many problems can be avoided with adequate siting and layout considerations, particularly in regard to ground failure. Chapter VI presents design considerations which can provide significant protection to equipment. Design criteria for a variety of equipment types are given in Chapter VII. These are in the form of a suggested design code. The chapter includes a discussion of the background of each criterion and provides example calculations. Chapter VIII presents important concepts in emergency response and planning. Most of the techniques discussed were successfully proven in past earthquakes. Problem areas that require attention are also covered.

CHAPTER II

PHYSICAL MANIFESTATIONS OF EARTHQUAKES

The purpose of this chapter is to provide a very basic description of seismic phenomena for those unfamiliar with earthquakes. If seismic analysis is planned by such readers, it is recommended that more in-depth information be obtained from readily available earthquake engineering texts (001).

A. CAUSES OF EARTHQUAKES

The current theory for the cause of earthquakes is based on plate tectonics. The earth is composed of a solid core surrounded by a dense liquid magma on which a crust floats. The crust varies in thickness and density; the oceanic plates are more dense and thinner than the continental plates. The crust is divided into a number of tectonic plates, e.g., North American, Pacific, etc., which are continually moved by convection currents within the liquid on which they float. These plates can interact in three different ways. Subduction occurs when a more dense ocean plate is pushed under a less dense continental plate. Two continental plates can push together such as the Indian and Asian plates forming the Himalayas. A strike slip occurs when two plates move past one another. This is the situation forming the San Andreas fault, where the Pacific plate is moving northwest and sliding by the North American plate. The relative movement between these two plates is about 1-3/4 inches per year. The most devastating earthquakes occur along the plate boundaries. Figure II-1 delineates lines of major plate interaction.

Intraplate earthquakes, which are caused by crustal thinning, magma implacement or a spreading center, can also occur. Earthquakes in the central and eastern United States are typical of these. The Sierra Nevada mountains were formed in this manner. The most severe series of earthquakes in the history of the United States (1811 and 1812) in what is now New Madrid, Missouri, were of this origin.

As movement between plates occurs, the material in contact can deform plastically with no sudden release of energy (creep), or stress can build up to the point where the material yields, giving off a sudden release of energy. The latter is most common in brittle rock.

The location in the crust where calculations indicate the first seismic wave to have originated is referred to as the focus or hypocenter. The epicenter is directly above on the earth's surface. The fault plane is the plane along which the crust breaks.

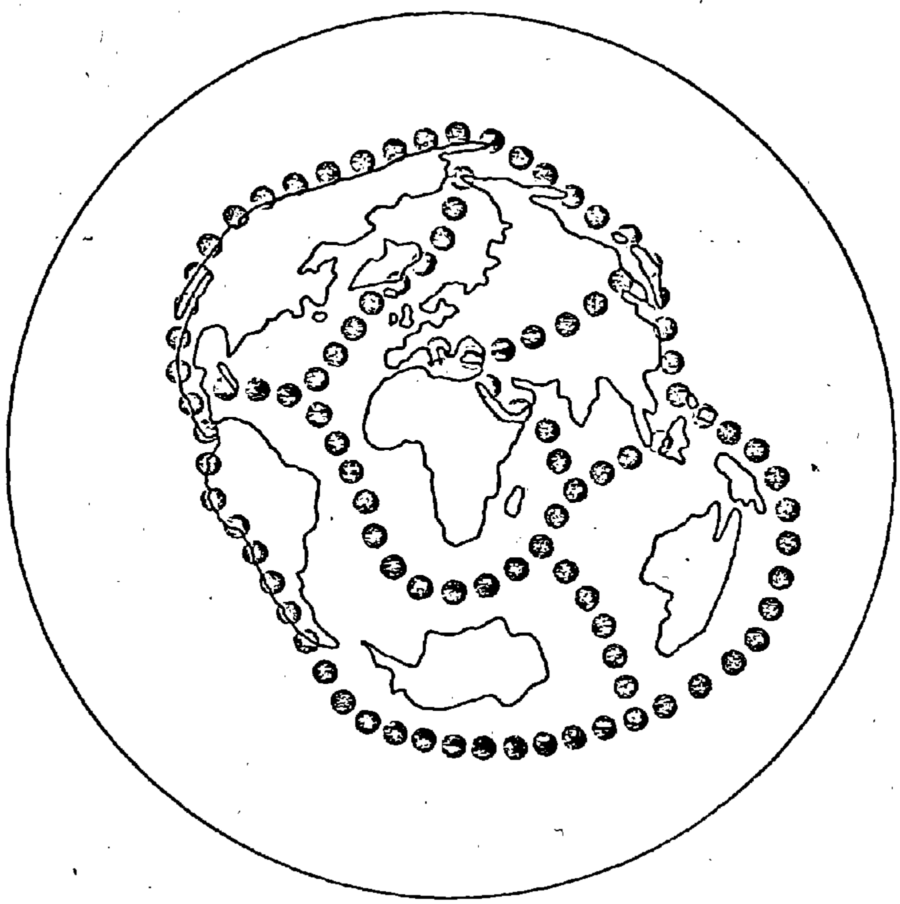


Figure II-1. Lines of major tectonic plate interaction (002).

B. MEASUREMENT

Earthquakes can be measured in several different ways. The Richter magnitude represents the log of the amount of energy released during an earthquake. Each increase of 1 on the open-ended Richter scale represents a 10-fold increase in the amount of energy released. The largest known Richter magnitude ever experienced was estimated at 9 in Lisbon in 1755. The Richter magnitude, however, is not necessarily representative of the extent of damage.

The Modified Mercalli intensity scale, on the other hand, is based on observed damage. The Modified Mercalli scale, as shown on Table II-1, is close-ended with a maximum of 12. The 1811/1812 New Madrid earthquake approached this maximum value.

The Modified Mercalli scale and the Richter scale are not directly related, although attempts have been made to relate them. A shallow earthquake with a low Richter magnitude may have a high Modified Mercalli intensity near the epicenter. By the same token, a deep earthquake with a high Richter magnitude may have little associated damage and therefore a low Modified Mercalli intensity.

Other countries, notably Japan, have earthquake intensity scales with different sets of descriptions.

TABLE II-1. MODIFIED MERCALLI INTENSITY SCALE

I. Not felt except by a very few under especially favorable circumstances.

II. Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing.

III. Felt quite noticeably indoors, especially on upper floor of buildings, but many people do not recognize it as an earthquake. Standing motorcars may rock slightly. Vibration like passing truck. Duration estimated.

IV. During the day felt indoors by many, outdoors by few. At night some awakened. Dishes, windows and doors disturbed; walls make creaking sound. Sensation like heavy truck striking building. Standing motorcars rocked noticeably.

V. Felt by nearly everyone; many awakened. Some dishes, windows, etc., broken; a few instances of cracked plaster; unstable objects overturned. Disturbance of trees, poles, and other tall objects sometimes noticed. Pendulum clocks may stop.

VI. Felt by all; many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight.

VII. Everybody runs outdoors. Damage negligible in buildings of good design and construction; slight to moderate in well built ordinary structures; considerable in poorly built or badly designed structures. Some chimneys broken. Noticed by persons driving motorcars.

VIII. Damage slight in specially designed structures; considerable in ordinary substantial buildings, with partial collapse; great in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Persons driving motorcars disturbed.

IX. Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb; great in substantial buildings, with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken.

X. Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations; ground badly cracked. Rails bent. Landslides considerable from river banks and steep slopes. Shifted sand and mud. Water splashed (slopped) over banks.

XI. Few, if any (masonry), structures remain standing. Bridges destroyed. Broad fissures in ground. Underground pipelines completely out of service. Earth slumps and land slips in soft ground. Rails bent greatly.

XII. Damage total. Waves seen on ground surfaces. Lines of sight and level distorted. Objects thrown upward into the air.

C. MOTIONS

When an earthquake takes place, energy is released in the form of mechanical waves radiating away from the source. Waves take three forms. Primary waves (P) are compressive waves (e.g., sound waves), which travel at approximately 3 to 4 miles per second. Secondary (S) or shear waves vibrate normally to the direction of propagation; these propagate at about 2 miles per second. Surface or Love waves (L) are a combination of the other two; these are normally quite small. Note that because of differences in velocity, a specific location will experience primary waves first, followed by secondary waves.

As shaking is transmitted from an earthquake focus to a particular site, the characteristics of the shaking are modified. Factors that affect this transformation include:

(1) Depth of the focus.

(2) Type and distance of intervening crustal formations.

(3) Type and depth of overlying soil strata at the site.

Four broad categories of earthquakes can be given:

(1) Primarily a single shock. This results from shallow earthquakes and occurs on firm ground at short distances from the epicenter.

(2) Moderately long, irregular motion. This occurs on firm ground at moderate distances from the epicenter. Most earthquakes of concern fall into this category.

(3) Long ground motions exhibiting prevailing periods of earth shaking. This type is associated with multiple layers of soft soil.

(4) Ground motions involving large-scale, permanent deformations of the ground. This type results in large-scale landslides or soil liquefaction.

Of the types of earthquake shaking, the second is the one addressed primarily by the analytical procedures discussed in this report.

Methods for analyzing earthquake ground motion consist of first assessing the attenuation from the assumed earthquake epicenter to the site through crustal rock, then assessing the filtering of the horizontal shear waves through the various levels of softer stratified soil through successive reflections and refractions. The resulting surface motions can then be used to analyze the response of man-made systems. For a particular site, this filtering may indicate that high frequency motions are minimized such that the response of structures with short periods of vibration may be less on soil than on rock. Similarly, structures having long periods of vibration (such as open water tanks) may have their motions amplified.

In the analysis of structural systems, one useful tool is the development of spectral responses of elastic single degree-of-freedom (SDOF) systems on a rigid base to anticipated ground motions. With any given ground motion, the peak or maximum responses (deformation, velocity and acceleration) can be obtained for such a system having all periods of vibration within the range of interest. A plot of these maxima constitutes the SDOF spectra for that ground motion. SDOF spectra can be obtained assuming the systems to have varying amounts of damping. Thus, spectra are usually developed for 1/2%, 2%, 5%, 10%, and 20% of the critical damping of the system analyzed. The curves thus developed are irregular in shape and vary considerably based on the content of the ground motion. To arrive at a design device, these spectra can be modified to smoothed spectra which represent the best judgment from a number of spectral plots.

D. SURFACE FAULTING

Surface faulting can be evidenced by either vertical or horizontal motion of adjoining blocks. When the terrain is broken in a series of elevated blocks, these are termed horsts. These may be separated by depressed blocks or grabens. A long narrow graben is termed a rift. Small scale vertical motion features are termed dip-slip displacements, resulting in scarps and scarplets.

Fault traces resulting from horizontal motion of adjoining blocks are called strike-slip. These can be evidenced by offsets of features (such as streams), parallel drag cracking, chevron cracking, open cracks and fissures, and by what appear to be giant mole tracks.

Movement along fault traces can be sudden, such as that causing larger earthquakes, or can be very slow; slow motion is called creep.

Designing to overcome the effects of surface faulting is not normally accomplished structurally. Rather, systems which may be subjected to potential damage from faulting are designed as redundant systems, having multiple facilities in different geologic settings.

E. GROUND FAILURE

Seismic motions commonly cause ground failure, including consolidation, liquefaction and landslides.

Earthquake motions can consolidate soils in much the same way as a vibratory roller consolidates sand at a construction site. Fill areas and deltaic deposits are particularly vulnerable. Structures supported by the soil that is consolidated may settle unevenly.

Many areas experience a potential loss in soil support capability during an earthquake by the phenomenon known as soil liquefaction. The soil temporarily behaves as a liquid that provides little or no resistance against translation or rotation of a supported structure.

Liquefaction of a supporting soil mass lying on a slope can lead to landslides. Landslides can develop when any unstable ground (such as material loosened by weathering) on a slope becomes shaken loose.

More detailed information on the various forms of ground failure is presented in Chapter V.

CHAPTER III

FUNCTIONAL ANALYSIS OF WATER AND SEWAGE SYSTEMS

Properly functioning water and sewage systems are vital for the physical and economic health of our society. Each day, these systems provide billions of gallons of potable drinking water and collect and treat similar amounts of sewage. Most treatment facilities are very large when compared to industrial chemical processing plants. For example, a 10 million gallon per day water treatment plant is considered to be small. However, its capacity expressed in typical industrial production terms is over 15 million tons per year. Few, if any industrial plants exceed this production level. Water and sewage treatment systems are also complex. For example, a typical secondary sewage treatment plant will contain over 200 different pieces of equipment.

At the present time, water and sewer systems, with some exceptions, do not provide special seismic protection beyond that required in building codes. These codes, as presently constituted, are primarily aimed at structural elements in buildings and do not adequately address other vital components of water and sewage systems.

The ideal condition for insuring effective operation of these systems is for all elements to receive the highest possible protection. However, this is not normally economically feasible, particularly in this period of rapidly rising costs of providing water and sewer service due to more stringent environmental regulations. The utility manager and designer, then, must decide where to place emphasis in seismic strengthening to obtain the most protection for available funds.

This chapter presents a procedure whereby the utility manager or designer can focus attention on critical elements so as to maximize performance after earthquake events. The procedure includes the following general steps:

- Outline system operating/performance goals
- Identify all functions that must be performed to meet the goals
- Identify equipment normally used to perform the required functions
- Identify performance goals for individual equipment items
- Identify alternative means of performing functions

This procedure can be used to identify pieces of equipment needed for both normal and emergency operation. The equipment identified can then be characterized as to location, interrelationships with other elements, and required level of availability and performance. This will provide the rationale for seismic design.

The examples used in this chapter are based on two operating plants: Dalecarlia Water Treatment Plant (Washington, DC) and the Flint, Michigan sewage treatment plant. These plants were selected because they represent typical installations and data on actual design were available or readily obtainable.

A. PERFORMANCE GOALS

There are many approaches to formulating performance goals for public water and sewage systems in relation to earthquakes. In this discussion, a set of goals is suggested for illustrative purposes. However, specific goals for each utility must be tailored to meet particular local needs. Therefore, performance goals should be developed by each utility. In order to be effective, they should be developed in cooperation with public health officials, local fire officials, and representatives of other major affected groups (e.g., industries).

The overall goal of all water and sewage systems is to protect the health and safety of the public. Secondary goals are to support economic activities and provide sufficient services to maintain the accepted lifestyle of the general population.

Public water supplies are used for a variety of activities:

- Drinking
- Cooking
- Bathing
- Cleaning
- Flushing toilets
- Fire fighting
- Industrial processes
- Commercial establishments
- Cooling
- Street cleaning
- Lawn watering

To support all of these activities, the delivered water must be free of pathogenic organisms, free of pollutants, aesthetically pleasing (taste, odor, color), and in sufficient volume and at sufficient pressure to meet demands of users.

Public sewage systems must be capable of rapidly collecting all wastes, treating the sewage to a degree appropriate to protect public health and protect the aquatic environment, and discharging the treated wastes at a safe location.

Of the above activities, the following are judged to be most important to protect public health and safety:

- Water for drinking and cooking
- Water for firefighting
- Sewer capacity to remove water used

Drinking and cooking require minimal amounts of water in relation to normal per capita use (100-125 gal/cap/day). However, firefighting water requirements are very high and not readily predicted. Required sewerage system performance is less well defined. If, after an earthquake, the water system is capable of providing only enough water for drinking and cooking and firefighting, then there will be little immediate need for sewerage capacity (no sewer discharge from firefighting). However, if the water system is only lightly damaged or recovery is rapid, the sewage system must be able to collect all discharged waste in order to avoid public health problems. To date, little emphasis has been placed on seismic protection of sewage systems.

Another important aspect of performance goals is provision for gradual recovery of the systems. It is assumed that recovery operations will begin shortly after the earthquake event. Some temporary repairs (i.e., "makeshift operations") will be required to keep the systems in operation. Other repair activities will commence within a few days of the earthquake, focussing on critical system elements. Some repairs may take several months to complete.

Based on the above discussion, the following operating goals are suggested. They are concerned primarily with the treatment plant and pumping stations which are the focus of this report but could readily be extended to pipeline elements.

WATER

a. Primary goals during the immediate emergency period (first 24 hours)

   1. Continuous hydraulic flow through or around the plant
   2. Provide disinfection for public health
   3. Provide for safety of personnel in the plant

b. Secondary goals - recovery period

   1. Makeshift operations on certain equipment will be acceptable for recovery period of up to 6 months
   2. Emphasis to be placed on rapidly upgrading quality of treated water - first filtration, then coagulation - finally aeration and treatment for stabilization. Sludge handling to be reinstituted with coagulation.

c. Shutdown philosophy

   Based on the above, certain unit operations could be bypassed or shutdown for the short term (up to 2 weeks): coagulation and settling; filtration and residue treatment and disposal. These are probably best accomplished by temporary bypasses of the entire unit operation. Other unit operations could be shut down for a longer term (up to 6 months) without serious public health or safety implications. The main impact of their shutdown would be aesthetics and economics. They include: aeration and stabilization.

d. Critical elements

The elements or systems that must be protected to meet basic goals are those that allow full hydraulic plant capacity and adequate disinfection: receiving raw water, disinfection, and discharging treated water. The other systems in the plant could be bypassed. If bypassing is not practicable, then the hydraulic capacity of all flow elements in each of the other systems must be protected. As a matter of basic philosophy, the provision of built-in bypass facilities around each nonessential unit operation is favored. The provision of bypasses around water treatment operations must be cleared with appropriate public health agencies.

e. Makeshift operations

For water systems, a number of elements (and systems) are judged to be capable of functioning under makeshift conditions for up to six months. "Makeshift" includes temporary repairs, provision of alternate elements or equipment to perform the necessary function, provision of equipment or arrangements that would not be considered cost-effective over a long period of time because of increased need for operator attention, maintenance, rapid wear, etc. While makeshift operations will be discussed, it is important that each plant review important elements or equipment in each system as part of earthquake protection planning to determine beforehand the possible makeshift operations and methodologies that may be effective and to have or know the location of required supplies and equipment.

SEWAGE

a. Primary goals

Same as water.

b. Secondary goals

1. Same as water.

2. Emphasis is to be placed on rapid restoration of treatment capability - first primary treatment and sludge handling. For purposes

of this study, secondary treatment is not judged to be as critical.

c. Shutdown philosophy

Based on the above goals, certain whole operations can be shut down for the short term (up to 2 weeks): primary treatment and sludge handling and disposal. The goal of hydraulic integrity around the entire plant is probably best handled through bypasses around the primary and secondary treatment systems. Secondary treatment could in many cases be shut down for a longer term (up to 6 months). However, this must be based on an analysis of the public health implications of a partially treated discharge to the specific receiving water of interest. Sludge treatment and disposal subsystems must be protected such that they will be capable of effective performance when the primary and secondary system is operable. Otherwise, the primary and secondary system cannot be used to remove pollutants.

d. Critical elements or systems

The elements or systems that must be protected to meet basic goals are those that allow full hydraulic plant capacity and adequate disinfection. The basic system includes: head works, disinfection and discharge of treated effluent. The other systems in the plant could be shut down (for 2 weeks to 6 months as described above). If bypassing is not practicable, then the hydraulic capacity of all flow elements in each system must be protected. As a matter of basic philosophy, the provision of built-in bypass facilities around each nonessential unit operation is favored. This does not consider, however, the existing regulatory policies of public health agencies in relation to system bypasses and discharge of partially treated effluent during emergency situations.

e. Makeshift operations

Same as for water.

Note that the above goals are based on systems or equipment operating up to normal standards. Other approaches, such as defining acceptable treated water quality or percent of population able to be served could also be used. The approach suggested was selected because it meshes well with design approaches used for seismic protection.

B. FUNCTIONAL ANALYSIS

Functional analysis has been extensively used in reliability evaluations of military systems. It is useful in that it promotes a better understanding of the interrelationships of elements in complex systems. The approach is based upon statements of function or purpose. Functions are expressed as a verb (or action) and object, for example "provide water supply." Functional analysis begins with a statement of the highest system function. Second level functions are then defined. These are functions required to perform the first level function. The second level functions are then each broken down further into subfunctions which, in turn, are required to perform the second level functions. This procedure is repeated until the desired level of detail is reached.

Once the array of functions is complete, one can begin to relate functions to equipment and/or operations. Thus, functional analysis is a tool for identifying the equipment required in the design of complex systems. One can also identify alternative means of satisfying functions. For example, a given function might be performed by highly sophisticated automatic equipment, low technology equipment with significant operator requirements or little or no equipment with large manpower needs.

For this project, functional analysis has been performed for illustrative purposes for both water and sewage systems. Figures III-1 and III-2 show three levels of functions for water and sewage systems, respectively. The third level functions given in these figures are further broken down to the fourth level <u>for treatment</u> in Tables III-1 and III-2 for water and sewage systems, respectively. Third and fourth level functions are given in the first two columns in these tables. Some fourth level functions are further broken down where warranted.

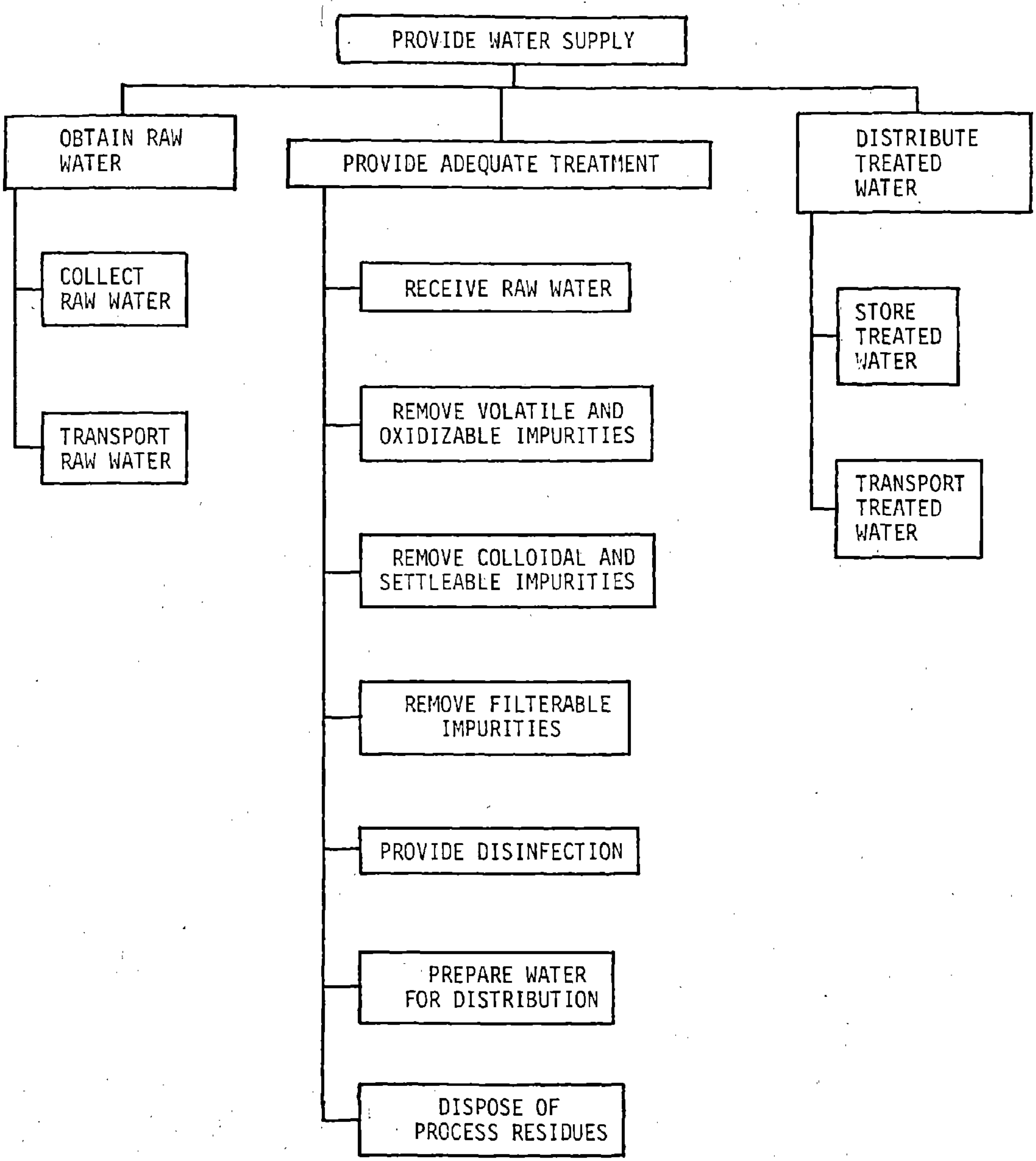


Figure III-1. Functional staging diagram - water supply.

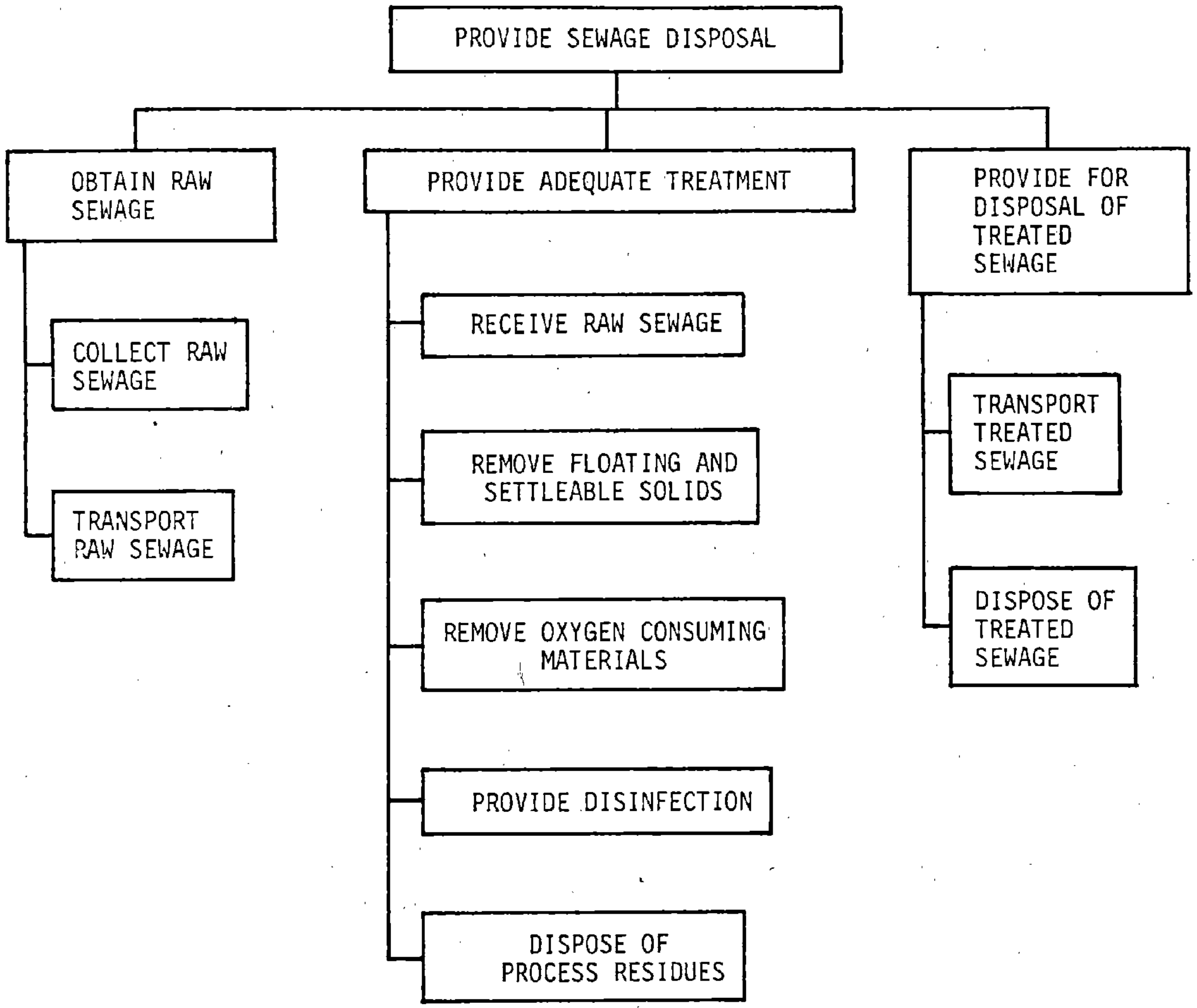


Figure III-2. Functional staging diagram - sewage disposal.

For each of the lowest level functions, corresponding process equipment normally used is identified. For this report, conventional complete water treatment and secondary sewage treatment are included. Unit operations included are:

Water treatment

- Head works
- Aeration
- Coagulation
- Settling
- Filtration
- Disinfection
- Clear well storage
- Residue disposal

Sewage treatment

- Head works
- Screening, grit removal, comminution
- Primary treatment
- Secondary treatment
- Disinfection
- Sludge treatment and disposal

The list of process equipment is based upon existing plants (verified through plant visits) and is considered typical of practice. One might wish to add or subtract unit operations or change specific equipment to better describe conditions at a particular treatment installation.

Tables III-1 and III-2 also include information that relates to seismic design of equipment. The typical location of each equipment item is listed according to the following key:

A. Buried or in-ground

B. On-ground (surface)

C. Elevated

D. Building or structure supported

The interrelationships with other elements is another important design factor. They are listed for each equipment item as follows:

I. Isolated (not pipe)

II. Interconnected, but separate

III. Mechanically coupled, not separate

IV. Connecting piping

Isolated equipment is that which stands alone and is not structurally connected to any other equipment. An example would be a storage bin for chemicals. The second category refers to that equipment which is not mechanically or structurally coupled, yet is interconnected with other process elements by piping or other equipment. An example of mechanically coupled equipment is an air meter which is coupled to an air compressor. The last category refers to the piping (and valves) that connect various equipment items.

Required availability ratings are also included in Tables III-1 and III-2 for each equipment item. These are used in design and planning to identify those equipment items that are critical and, therefore, require additional protection. The ratings given in the tables are listed below:

1. Continuous operation is essential to achieve performance goals.
2. Continuous operation is preferable to other means of functioning (e.g., "jury rig").
3. Short term (24 hours) makeshift operation is acceptable.

4. Short term (24 hours) shutdown is acceptable.
5. Long term makeshift operation is acceptable (up to 6 months).
6. Intermediate shutdown is acceptable (up to 2 weeks).
7. Long term shutdown is acceptable (up to 6 months).

These ratings are used to develop seismic coefficients discussed in Chapter VII.

Note that those equipment items with a required availability rating of 1 or 2 are considered essential and require special attention. They are those associated with maintaining flow through the treatment plant and providing for adequate disinfection. Required availability ratings are also given for entire functions. This was done to provide an indication of which functions are critical to meet overall performance goals. Note that within many subfunctions rated at 7, which indicates that long term bypassing may be acceptable, some equipment items are rated at 1 or 2. This was done under the assumption that function (or unit operation) bypassing would not be allowed. If bypassing is allowed by regulatory agencies, all equipment in that function (or unit operation) would then have a lower, less critical availability rating.

The information given in Tables III-1 and III-2 is for illustration purposes. Each plant engineer or planner should evaluate functions and conditions specifically for the plant in question.

TABLE III-1. FUNCTIONAL ANALYSIS OF WATER TREATMENT WITH LISTING AND DESCRIPTION OF ASSOCIATED PROCESS EQUIPMENT*

Function	Subfunction	Process equipment	Location	Relation to other elements	Required availability rating
Receive raw water					1 or 2
	Receive raw water from transmission pipelines	Influent pipes, valves	A	IV	1 or 2
	Establish hydraulic head	Influent well	A	II	1 or 2
	Provide for plant bypass	Bypass chamber	A	II	1 or 2
		Bypass piping and valves	A	IV	1 or 2
	Convey water to next operation	Pipes and valves	A	IV	1 or 2
	Monitor flow	Flow meter and recorder	D	II	1 or 2
Remove volatile and oxidizable impurities					7
	Receive influent flow	Raw water channels or pipes	A	IV	1 or 2
	Contain flow and provide turbulence	Aeration tank	A or B	II	1 or 2
		Filters (air)	B	II	7
		Air compressors	B	III	7
		Air meters	B or D	III	7
		Piping (air)	B	IV	7
		Diffusers	B or D	II	7
	Convey for further treatment	Piping and valves	A	IV	1 or 2
Remove colloidal and settleable impurities					5 or 6
	Receive influent flow	Piping and valves (raw water channels)	A	IV	1 or 2

* See text for explanation of symbols.

TABLE III-1 (continued)

Function	Subfunction	Process equipment	Location	Relation to other elements	Required availability rating
Remove colloidal and settleable impurities (continued)					
	Provide chemicals	Aluminum sulfate: hopper cars, drums, paper bags	B or C	I	5 or 6
		Storage bins (with vibrators)	C	I	5 or 6
		Dry-feed machines	B	I	5 or 6
		Piping	D	IV	5 or 6
	Provide for mixing	Mixing tank	A	II	1 or 2
		Mixer	D	I or III	5 or 6
	Contain chemically treated water	Flocculation-sedimentation basin	A	II	1 or 2
		Flocculating baffles	D	III	5 or 6
	Collect and convey settled water for further treatment	Overflow wiers and effluent channels	D	III	1 or 2
		Piping and valves	A	IV	1 or 2
	Collect and convey settled solids	Sludge scraper and drive	D	III	5 or 6
		Sludge pumps	B	II	5 or 6
		Sludge control valves and piping	A or B	IV	5 or 6
		Meter (sludge)	D	II or III	7
Remove filterable impurities					5 or 6
	Receive settled water	Pipes, valves and fittings	A or B	IV	1 or 2
	Contain settled water and provide filtration	Filtration tank	A,B or C	II	1 or 2
		Distribution pipes	D	IV	1 or 2
		Valves and motors	B	III	1 or 2
		Filter media	D	III	5 or 6
		Rate controllers	D	III	5 or 6
		Flow meter	D	III	7
		Underdrains	D	III	1 or 2
	Collect filtered water	Flow collection chamber	D	III	1 or 2
	Monitor filter condition	Head loss meter	D	III	5 or 6

TABLE III-1 (continued)

Function	Subfunction	Process equipment	Location	Relation to other elements	Required availability rating
Remove filterable impurities (continued)	Collect filtered solids	Washwater pipe and valves	D	IV	5 or 6
		Backwash water trough	D	III	5 or 6
		Surface wash pipe and sweep mechanism	D	II	7
	Convey filtered water	Filtered water pipe and valves	D	IV	1 or 2
	Convey filtered solids	Drain pipes and valves	D	IV	5 or 6
Provide disinfection					1 or 2
	<u>Store chlorine</u>				
	Provide for chlorine container storage	Storage racks	B	I	1 or 2
	Provide mobility of container	Trolley hoist	B or D	I	5 or 6
	Monitor supply on hand	Weighing scale	B	II	3
	<u>Feed chlorine</u>				
	Supply chlorine	Chlorine cylinders	B or C	II	1 or 2
		Chlorinators	B	II	1 or 2
	Control feed	Flow control valves and piping (chlorine)	D	IV	1 or 2
		Flow control valves and piping (water)	D	IV	1 or 2
	Monitor and record chlorine feed rate	Recorder	D	II	5 or 6
	Analyze wastes for chlorine residual	Chlorine residual analyzer and recorder	B or D	II	5 or 6
		Ambient chlorine detector	D	I	1 or 2
	Convey chlorine solution	Piping and valves	D	IV	1 or 2
		Protected water system (piping, tank and pump)	D	II and IV	1 or 2

TABLE III-1 (continued)

Function	Subfunction	Process equipment	Location	Relation to other elements	Required availability rating
Provide disinfection (continued)					
	<u>Provide contact</u>				
	Receive effluent from preceding process	Pipe or channel	A or B	IV	1 or 2
	Contain flow	Contact tank	A or B	II	1 or 2
	Provide mixing	Chlorine diffuser	B or D	II or III	1 or 2
		Baffles	D	III	5
	Convey treated water	Disinfected water pipe	A	IV	1 or 2
	Meter or monitor flow	Meter	A,B or D	II	5
Prepare water for distribution					1 or 2
	Receive flow	Pipes and valves	A	IV	1 or 2
	Provide storage	Finished water reservoir	A or B	II	1 or 2
	Remove and convey water for distribution	Finished water pumps and motors	B or D	II	1 or 2
		Pump motor controls	B or D	II	1 or 2
		Finished water piping and valves	A	IV	1 or 2
	Monitor flow	Flow meter and recorder	D	II	6
Dispose of process residues					5 or 6
	<u>Dewater solids</u>				
	Receive solids	Influent pipes	A or B	IV	5 or 6
	Contain solids	Thickening tank	A or B	II	5 or 6
		Piping and valves	A,B or D	IV	5 or 6
	Convey solids for further treatment or discharge	Piping and valves	A or B	IV	5 or 6

TABLE III-2. FUNCTIONAL ANALYSIS OF SEWAGE TREATMENT WITH LISTING AND DESCRIPTION OF ASSOCIATED PROCESS EQUIPMENT *

Function	Subfunction	Process equipment	Location	Relation to other elements	Required availability rating
Receive raw sewage					1 or 2
	Receive sewage from interceptors	Raw sewage mains	A	IV	1 or 2
	Establish hydraulic head	Pump station wet well	A	II	1 or 2
		Sewage pumps and motors	B	II	1 or 2
		Pump motor controls	D	II	1 or 2
		Piping and valves	D	IV	1 or 2
	Monitor flow	Flow meter and recorder	D	II	6
	Stabilize hydraulic head	Influent pipes	A	IV	1 or 2
		Head works chamber	A	II	1 or 2
		Overflow weirs	A	III	1 or 2
		Overflow channel	A	III	1 or 2
		Sluice gate or valves	B	III	1 or 2
		Head works effluent pipe	A	IV	1 or 2
	Provide for plant bypass	Bypass chamber	A	II	1 or 2
		Bypass pipe	A	IV	1 or 2
	Monitor flow	Flow meter	B	II	6
Remove floating and settleable solids					Process- 5 or 6 Hydraul.-1 or 2
	<u>Remove gross solids</u>				
	Receive raw sewage	Bar screen influent pipes	A or B	IV	1 or 2
	Remove or alter large solids	Bar screens (and rake)	B	III	5
	Remove or convey grit	Sewage flow control (diversion box)	A	III	2 or 5
		Grit chamber	A	II	2 or 5
		Grit removal mechanism	D	III	5
		Grit pumps	A	II	5
		Influent and effluent piping (grit chamber)	A or B	IV	2 or 5

* See text for explanation of symbols.

TABLE III-2 (continued)

Function	Subfunction	Process equipment	Location	Relation to other elements	Required availability rating
Remove floating and settleable solids (continued)		Weirs	A	III	5
		Air compressors	A or B	II	7
		Air piping and valves	D	IV	7
		Air drops and diffusion	D	II	7
	Break up remaining solids	Comminutor chambers	A,B or C	II	2 or 5
		Comminutors	D	II	6
		Control and bypass devices (stop plates, valves, etc.)	D	II or III	2 or 5
		Comminutor influent and effluent piping or channels	A or B	IV	1 or 2
	Remove settleable and flotable solids				
	Receive and control wastes for settling	Influent pipes or channels	A or B	IV	1 or 2
		Distribution pipes for all wastes to be settled	A or B	IV	1 or 2
		Settling tanks	A or B	II	1 or 2
		Control valves and gates	A or B	III	1 or 2
		Meter (sewage flow to each clarifier)	D	II or III	7
	Collect settled solids	Sludge scraper and drive	D	III	5 or 6
	Convey settled solids	Sludge pumps	B	II	5 or 6
		Control valves (sludge)	B	II	5 or 6
		Piping (sludge)	A	IV	5 or 6
		Meter (sludge)	D	II or III	7
	Collect floating solids	Scum collection mechanism	D	III	5 and 6
		Scum collection trough	D	III	5 and 6
		Scum removal pump (ejector)	A or B	II	5 and 6
		Scum control valves, gates, etc.	A or B	II	5 and 6
	Convey floating solids	Scum piping in settling tanks	D	IV	5 and 6
		Scum piping to scum disposal	A or B	IV	5 and 6
		Scum valves	A or B	II	5 and 6
	Collect and convey clarified wastes for further treatment	Primary treatment effluent channel	A or B	IV	1 or 2
		Effluent valves and gates	A or B	II	1 or 2
		Overflow weirs	D	III	5 and 6

TABLE III-2 (continued)

Function	Subfunction	Process equipment	Location	Relation to other elements	Required availability rating
Remove oxygen consuming material					7
	Receive partially treated sewage				
	Convey influent from primary treatment	Channel or pipe (primary effluent)	A or B	IV	1 or 2
	Distribute flow to each aeration tank	Flow control valves and piping (primary effluent)	A or B	IV	1 or 2
	Monitor flow	Flow meter	D	III	7
	Provide aerobic contact between sewage and suspended microorganisms				
	Contain mixed liquor	Aeration tanks	A or B	II	1 or 2
	Generate air supply	Blowers	B	II	7
		Filter (air)	B or D	II or III	7
		Pump (cooling water)	B	II	7
	Monitor and control air supply	Meter (air)	D	II	7
		Flow control valves (air)	D	II	7
	Convey and diffuse air to mixed liquor tank	Diffusers (air)	D	II	7
		Piping (air)	D	IV	7
	Convey and distribute return sludge	Flow control valves (return sludge)	A or B	II	7
		Pumps	A or B	II	7
		Piping	A or B	IV	7
	Monitor and control sludge return rate	Meter (return sludge gpm)	D	II	7
	Control foam	Foam spray system (pump, pipe, nozzles, etc.)	A,B or D	II and IV	7
	Remove settleable and floating solids				
	Receive and distribute mixed liquor	Influent piping or channels	A or B	IV	1 or 2
		Distribution piping or channels	A or B	IV	1 or 2
		Meter	D	II or III	7
	Control flow to settling tanks	Control valves (mixed liquor)	A or B	II	1 or 2

TABLE III-2 (continued)

Function	Subfunction	Process equipment	Location	Relation to other elements	Required availability rating
Remove oxygen consuming material (continued)					
	Contain wastes for settling	Tanks (final clarifiers)	A or B	II	1 or 2
	Collect settled solids	Sludge scraper mechanism	D	III	7
	Collect floating solids	Scum removal mechanism	D	III	7
		Scum removal pump/ejector	A or B	II	7
	Convey waste sludge to sludge treat.	Piping (settled sludge)	A or B	IV	7
		Pump (sludge)	A or B	II	7
	Monitor waste sludge flow rate	Meter (sludge)	D	II	7
	Convey floating solids to sludge treat.	Piping (scum)	A or B	IV	7
	Convey clarified wastes for further treat.	Trough or pipe (secondary effluent)	A or B	IV	1 or 2
Provide disinfection					1 or 2
	Store chlorine				
	Provide for chlorine container storage	Storage racks	B	I	1 or 2
	Provide mobility of containers	Trolley hoist	B or D	I	5 and 6
	Monitor supply on hand	Weighing scale	B	II	3
	Feed chlorine				
	Supply chlorine	Chlorine cylinders	C	I	1 or 2
		Chlorinators	B	II	1 or 2
	Control feed	Flow control valves and piping (chlorine)	D	IV	1 or 2
		Flow control valves and piping (water)	D	IV	1 or 2
	Monitor and record chlorine feedrate	Recorder	D	II	5 or 6
	Analyze wastes for chlorine residual	Chlorine residual analyzer and recorder	B	II	5 or 6
		Ambient chlorine detector	D	I	1 or 2

TABLE III-2 (continued)

Function	Subfunction	Process equipment	Location	Relation to other elements	Required availability rating
Provide disinfection (continued)	Convey chlorine solution	Piping and valves	D	IV	1 or 2
		Protected water system (piping, tank and pump)	D	II and IV	1 or 2
	<u>Provide contact</u>				
	Receive effluent from preceding process	Pipe or channel	A or B	IV	1 or 2
	Contain flow	Contact tank	A or B	II	1 or 2
	Provide mixing	Chlorine diffuser	D	III	1 or 2
		Baffles	D	III	5
	Convey treated wastes to discharge or next treatment step	Piping (final effluent)	A or B	IV	1 or 2
	Meter or monitor flow	Meter (final effluent flow)	A,B or D	II	5
Dispose of process Residues					5 or 6
	<u>Receive waste solids</u>				
	Convey solids to primary dewatering facilities	Pumps	A or B	II	5 or 6
		Piping	A,B or D	IV	5 or 6
		Flow control valves	A,B or D	II	5 or 6
	Monitor flow	Meter (waste sludge flow)	D	II	5 or 6
	<u>Primary dewatering</u>				
	Receive solids	Influent pipe and valves	A or B	IV	5 or 6
	Provide conditioning chemicals (Ferric chloride, alum or lime)	(Granular) steel drums	B or C	I	6
		(Liquid) rubber lined tank cars; drums	B or C	I	6
		Solution storage tank, transfer pump and mixer	B,C or D	II	5 or 6
		Day tank	B or C	II	5 or 6

TABLE III-2 (continued)

Function	Subfunction	Process equipment	Location	Relation to other elements	Required availability rating
Dispose of process residues (continued)	Feed chemicals	Wet chemical feeder	B or D	II	5 or 6
	Convey conditioned solids for dewatering	Piping and valves	A or B	IV	5 or 6
	Contain solids for dewatering	Flotation thickening tanks	A or B	II	5 or 6
	Collect and remove dewatered solids	Sludge skimmer mechanism and controls	D	II	5 or 6
		Sludge well	D	III	5 or 6
		Grit screw mechanism and controls	D	II	5 or 6
		Grit scraper mechanism	D	II	5 or 6
		Grit well	D	III	5 or 6
		Pump (grit)	D	II	5 or 6
		Piping and valves	D	IV	5 or 6
	Provide means for dewatering solids (air)	Compressor	B or D	II	5 or 6
		Piping and valves	B or D	IV	5 or 6
		Filter (air compressor)	B or D	III	5 or 6
		Pressure tank	B or D	III	5 or 6
	Convey dewatered solids for further treatment	Pumps (sludge)	A or B	II	5 or 6
		Piping and valves	A or B	IV	5 or 6
		Meter	D	II	7
	Collect and convey excess water to disposal	Subnatant well	D	III	5 or 6
		Pumps	D	II	5 or 6
		Piping and valves	A,B or D	IV	5 or 6
	<u>Primary stabilization</u>				
	Receive partially dewatered solids	Piping and valves	A or B	IV	7
	Monitor and record flow	Recorder (waste sludge flow-digested sludge flow)	D	III	7
	Contain solids	Tanks (digester)	B	II	7

TABLE III-2 (continued)

Function	Subfunction	Process equipment	Location	Relation to other elements	Required availability rating
Dispose of process residues (continued)	Provide necessary stabilization conditions	Piping and valves (waste sludge)	D	IV	7
		Heat exchanger	D or B	II	7
		Pump (recirculating-heater)	B	II	7
		Protected water system (tank, pipes, valves)	D	IV or II	7
	Provide proper degree of mixing	Pump (digested sludge transfer)	D or B	II	7
		Piping and valves	D	IV	7
	Monitor and record flow and conditions	Recorder, meters	D	II or III	7
	Remove solids for further treatment	Pumps (digested sludge)	B	II	7
		Piping	B or D	IV	7
		Valves	B or D	II	7
		Piping and valves (digester supernatant)	A,B or D	IV	7
	<u>Secondary dewatering of solids</u>				
	Receive partially dewatered solids	Piping and valves	A or B	IV	5 or 6
	Provide chemicals	$FeCl_3$-drums, tank cars	B or C	I	6
		Lime-bags, drums, tank cars	B or C	I	6
	Provide chemical storage	$FeCl_3$ tank, drain, transfer pump and mixer	B or C	II	5 or 6
		Lime storage bin with vibrator	C	I	5 or 6
	Feed chemicals	Lime conveyor, hopper and chutes	D	I	5 or 6
		Lime feeding mechanism	B	II	5 or 6
		Lime slaker	B	III	5 or 6
		Protected water system (tank, pipe and pump)	D	IV or II	5 or 6
		Lime solution tank	B or C	II	5 or 6
		$FeCl_3$ solution day tank	B or C	II	5 or 6
		Pump (lime solution)	B or D	II	5 or 6
		Pump ($FeCl_3$ solution)	B or D	II	5 or 6
		Piping	D	IV	5 or 6

TABLE III-2 (continued)

Function	Subfunction	Process equipment	Location	Relation to other elements	Required availability rating
Dispose of process residues (continued)					
	Control feed rate	Valves	D	II	5 or 6
	Monitor and record feed rate	Recorder	D	II	7
	Contain solids for conditioning	Tank (conditioning)	B or C	II	5 or 6
	Mix chemicals and solids	Mixer	D	II	5 or 6
	Convey conditioned solids to dewatering facilities	Piping	B	IV	5 or 6
	Contain solids for dewatering	Vacuum filter unit, including drum and media	B	II	5 or 6
	Provide means for dewatering solids	Vacuum pump unit, separator and piping	B or D	II	5 or 5
		Protected water system (pipe, tank, pump, etc.)	D	IV or II	5 or 6
	Convey excess water to disposal	Pump (filtrate)	B or D	II	5 or 6
		Piping (filtrate)	A,B or D	IV	5 or 6
	Convey dewatered solids for further treatment	Sludge conveyor belt	B,C or D	I	5 or 6
	Secondary stabilization				
	Receive dewatered solids	Conveyor (see above)	B,C or D	I	7
	Contain solids	Incinerator (with gas, seal, doors, etc.)	B	I	7
	Monitor and record flow in	Scale and recorder (filter cake wt.)	D	II	7
	Provide proper conditions	Auxiliary gas supply (piping, burners, etc.)	D	II	7
		Scrubber	D	III	7
		Pump (scrubber water)	B or D	II	7
		Piping (scrubber water)	D	IV	7

TABLE III-2 (continued)

Function	Subfunction	Process equipment	Location	Relation to other elements	Required availability rating
Dispose of process residues (continued)		Final effluent well (scrubber water)	A or B	II	7
		Rabble arms, motor and gears	B or D	II	7
		Burners	D	II	7
		Meters, controls and recorders	D	II	7
		Blower (shaft cooling)	B	II	7
		Induced draft fans	D	II	7
		Afterburner	D	III	7
		Air compressor (service air)	B	II	7
		Air filter (air compressor)	D	III	7
		Piping and valves (air)	D	IV	7
	<u>Collect and remove residue for ultimate disposal</u>				
	Collect residue	Ash tank	B	II	7
	Remove residue for ultimate disposal	Screw feeder	D	II	7
		Conveyor belt to point of loading (e.g., truck)	B,C or D	I	7

CHAPTER IV
POTENTIAL DAMAGE

The purpose of this section is to describe potential modes of earthquake induced failure in water and sewage system facilities. The modes of failure will be related to the causative earthquake motions discussed in Chapter II of this report. Designs to resist these failure modes will be discussed in Chapters V, VI and VII.

Documented damages sustained by water and sewer systems in past earthquakes are included to exemplify the potential damage modes. This information is supplemented by damage reports from similar installations found in industry, other utilities and commercial buildings. Table IV-1 lists the earthquakes and associated pertinent information referred to in this section. Table II-1 in Chapter II describes the Modified Mercalli Scale referred to in Table IV-1.

This chapter is divided into four sections, including Source and Discharge Facilities, Transmission, Distribution and Collection Systems, Treatment Systems and Storage Tanks.

TABLE IV-1 CHARACTERISTICS OF SELECTED MAJOR EARTHQUAKES

EARTHQUAKE	DATE	EPICENTER	RICHTER	MODIFIED MERCALLI	ACCELERATIONS	DAMAGED AREAS	COMMENTS
San Francisco, California (003)	April 18, 1906	38°N, 123°W	8.3	XI		San Francisco, San Jose, Santa Rosa, Agnew's Asylum, Hollister, Salinas	190-270 miles of surface faulting
Kanto Japan (004,005)	September 1, 1923	34.5N, 139.2E; near Tokyo	7.9 - 8.16			Severe: Kanagawa, Tokyo; also in Shizuoka, Yamanashi, and Nagano Prefectures	
Fukui Plain, Japan (006)	June 28, 1948		7.2				
Kern County, California (007,003)	July 21, 1952	35.0N, 119.0W; 110 Km NW of Los Angeles	7.7	XI		Major: Tehachapi, Arvin; also in L.A., Arvin, Taft, Bakersfield, Wheeler Ridge	14 miles of surface faulting 8/22: Aftershock near Bakersfield, Richter 5.8
San Francisco, California (008,003)	March 22, 1957	37.7N, 122.5W. San Andreas rift zone near Mussel Rock	5.3	VII	0.11g 7 miles from epicenter; 0.01g 40 miles from epicenter	Major: Daly City; Minor: San Francisco	
Skopje, Yugoslavia (009)	July 26, 1963	42.1°N, 21.4°E	5.4 - 6.6	VIII		City of Skopje	
Prince William Sound, Alaska (003,010)	March 27, 1964	61.1N, 147.5W; 75-80 miles east of Anchorage	8.4	X	Horiz.: ≤.16g Vert.: Substantially less than above	Anchorage, Valdez, Cordova, Kodiak, Seward, Whittier	Liquefaction, landslides caused major damage, due to 200-300' thick layer of blue "Bootlegger Cove" clay underlying Anchorage, covered with 10-30' of dense sandy gravel; 400-500 miles of surface faulting; many aftershocks; damage to supposedly earthquake resistant buildings

TABLE IV-1 (Continued)

EARTHQUAKE	DATE	EPICENTER	RICHTER	MODIFIED MERCALLI	ACCELERATIONS	DAMAGED AREAS	COMMENTS
Niigata, Japan (011,012)	June 16, 1964	38.4N, 139.2E; 70 Km North of Niigata	7.7		400 gals at Atsumi, 300 gals at Senami, 250 gals at Tsuruoka, 190 gals, Niigata, basement	Niigata City, Yamagata and Akita Prefectures	Soil conditions: Sand and silt estuary deposits often extending to great depths; extensive liquefaction
Tokachi-Oki, Japan (013)	May 16, 1968	142°35'E, 40°41'N	7.9			Northern Honshu and Southern Hokkaido-Aomori, Hachinone, Mutsu, Towada, Gonone, Aomeri Prefective	
Santa Rosa, California (014,015)	October 1, 1969	Near Santa Rosa	5.6	VII-VIII	0.03g or 28cm/sec^2 instruments were 28 miles away	Santa Rosa	Actually 2 earthquakes within 83 minutes, treated as one
San Fernando, California (016,017)	February 9, 1971	34°24'N, 118° 24'W	6.6		Horizontal: .75-1.04g	Granada Hills, Sylmar and Olive View areas of city of LA and adjoining areas in S. Fern.	Area affected is located on alluvial plain, soil is sandy loam, except for the Knollwood Area, which is on the foothills of the Santa Susana Mountains
Managua, Nicaragua (018)	December 23, 1972	Debatable: either right of the city or 50 Km NE	6.25		E-W 0.39g; N-S 0.34g; Vertical 0.33g	Managua; cities 15-20 Km away were not damaged	City located on Lake Managua, but no liquefaction; landslides limited by severe drought
Lima, Peru (019)	October 3, 1974	12.265S, 77.795W	7.5	VII			

TABLE IV-1 (Continued)

EARTHQUAKE	DATE	EPICENTER	RICHTER	MODIFIED MERCALLI	ACCELERATIONS	DAMAGED AREAS	COMMENTS
Haicheng, China (020)	February 4, 1975		7.3				
Friuli, Italy (021)	May 6, 1976	46.4°N, 13.1°E	6.5	VIII-IX	0.37g	Friuli region of Venezia-Giulia Sector, heaviest damage in towns north of Udine	Two strong aftershocks occurred on September 15, 1976, centered near the main shock, with magnitudes of about 6.0
Tangshan, China (020)	July 28, 1976	118.2°E, 39.4°N	7.8	XI	0.097g 153 Km from epicenter, 0.073g 157 Km from epicenter	Major damage in Tangshan, Tientsin, damage also in Ninghua, Luanhsien	
Miyagiken-Oki, Japan (022,023,024, 025,026)	June 12, 1978	142°10'E, 38°09'N 100Km E of Sendai	7.4	VIII	.2-.33g; .045g in basement 305 Km away; 23g on 19th floor 422 Km away	East coast of Northern Honshu about 300 Km north of Tokyo, Sendai City Miyagi Prefective	Low, flat alluvial plains, local allurial terraces, and steep bedrock terrain of moderate relief; liquefaction at several sites on coastal flood plain bordering Bay of Sendai; several thousand landslides
Imperial Valley, California (027,028,029)	October 15, 1979	32.64°N, 115.33°W	6.6		.40g at El Centro	El Centro and surrounding Southern California communities; Calexico, Mexicali	
Greenville (Diablo-Livermore), California (030,031, 032)	January 24, 1980	37.83°N, 121.79°W	5.5		0.02-0.15g on the ground	Livermore and Dublin	A second principal shock occurred 14 Km to the south on January 27, Richter magnitude 5.8.

A. SOURCE AND DISCHARGE FACILITIES

This discussion encompasses water intake structures in impoundments or other water bodies, submerged intake and discharge lines and diffusers, source access and wells. Transmission lines connecting the intake structure to the treatment facility will be discussed in the following section. While earth and concrete impoundments are mentioned, a detailed discussion of their failure modes and design is beyond the scope of this report. Above-ground, well related facilities such as well houses and pump systems will be discussed in conjunction with treatment facilities.

INTAKE STRUCTURES

Water intake structures, typically tower type structures located in water impoundments, are subject to failure from earthquake forces. The lateral inertia effect of the structure's mass and surrounding water may cause failure in shear at the base of the structure or in bending of the column. The foundations of these structures may be founded on unstable submerged strata vulnerable to displacement. Nearby landslides of unstable soil may damage these intake structures as well. In general, landslides from unstable, steep ground slopes are a major cause of earthquake induced damage to water intake structures as illustrated in the three examples below.

A landslide from an adjacent earth dam embankment caused outlet tower #1 in the Lower Van Norman reservoir (Los Angeles Department of Water and Power, LADWP) to topple during the 1971 San Fernando earthquake. Sand, gravel and rocks entered the distribution system through the broken intake, causing extensive damage to pumps, instrumentation and controls. Outlet tower #2 in that same reservoir experienced slight cracking. Both these towers, built in 1914-1915, were designed as unreinforced concrete gravity structures (033).

The 1972 Managua earthquake induced a landslide on the steep bank of Lake Asosoca. A pump station supplying the majority of the City of Managua's water was supported on piling extending into the lake. The pump suctions, located one meter off the lake bottom, were buried by the landslide, requiring excavation by divers (034,035).

During the 1964 Alaska earthquake, the water intake at the Eklutna hydroelectric power project failed due to movement of the fill on which a portion of the structure was founded (036).

SUBMERGED INTAKE AND DISCHARGE LINES AND DIFFUSERS

Soil deposits formed under submerged conditions may be unstable because of their low relative density. Submerged water intake lines, sewage outfall lines and diffusers, which are structurally similar, have often been built on submerged deposits, which are subject to liquefaction and landslides.

In the Eklutna Project in Alaska, earthquake-induced horizontal motion caused the precast concrete conduit connecting the intake structure to the power plant to separate at the joints. Debris entering through the open joints and broken intakes required extensive cleanup. The replacement intake structure was keyed into the glacial fill below, providing a more stable foundation (036). In the 1971 San Fernando earthquake, the outfall line from the intake tower in the Upper Van Norman Reservoir was damaged through either compression or extension of various joints (033).

IMPOUNDMENTS, DAMS AND APPURTENANT STRUCTURES

Although concrete and earthen dams may be integral components of water systems, their design is beyond the scope of this report. In the San Fernando earthquake of 1971, the Upper Van Norman Reservoir earthen dam moved 5 feet at the crest and suffered a three foot settlement. The intake structure of the Lower Van Norman Reservoir dam was destroyed by a landslide on the dam. The bypass reservoir in the complex suffered cracking of the asphalt lining (033).

SOURCE ACCESS

Surface water sources are typically impounded in the upper reaches of watersheds where possible pollution is limited. Access to these impoundments and related facilities is often provided by mountain roads, which may be subject to landslides in earthquake prone areas.

In the 1971 San Fernando earthquake, the Pacoima Dam access was blocked for several hours, due to a landslide which occurred 20 hours after the initial shock (033). An earthquake induced landslide on the side of Lake Asosoca in Managua destroyed the access road to the pumping station (037).

WELLS

Groundwater withdrawn through wells is the primary or secondary water source in many areas. Wells can be affected by earthquakes in a variety of ways. The well shaft can be crushed or sheared off by displacement of the ground across the shaft or by vibration of the ground. Ground displacements may disrupt the groundwater hydrology, decreasing or even cutting off water supply to an aquifer (038). Local soil disturbance from shaking may plug the well screen. The pump and piping may be damaged from relative movement between the units. Failure of local sewer lines or septic tanks permit sewage to leak into the aquifer, contaminating the water (039).

A well casing at the Port of Whittier was bent during the 1964 Alaska earthquake, making it difficult to remove the turbine pump. Consolidation of the strata during the earthquake caused some well casings to extend an additional six inches above the ground (040). Of seven wells used for high demand and emergency situations in Anchorage, two were lost completely; one was inoperable but repairable; two were operable but damaged; and two were undamaged. In the region of massive earthslides and liquefaction, pump lines were completely destroyed in two wells (041). Operation of two structurally undamaged wells was precluded by loss of emergency power (038). In the 1952 Kern County California earthquake, many wells located in an area of surface disturbance were damaged due to the lateral displacement of the upper end of the casing (042).

In the 1971 San Fernando earthquake, Los Angeles County experienced breaking of pipe connections at two wells, with horizontal displacement of one well cover. Damages to wells in the City of San Fernando are summarized below (033).

Well #1	Slight contamination of water from broken sewers
Well #2	Slight casing bending and rupture Pump pad and floor cracked Contamination from sewers and septic tanks
Well #3	Pumphouse floor badly broken Support blocks for discharge misaligned Contamination
Well #4	Cracking pumphouse floor slab Misalignment of discharge line support blocks Slight break in well casing Contamination
Well #5	Pump shifted in well
Well #7	Electrical cable split near top Many doglegs and twists, bulges Constriction - Well abandoned

The most wide-spread earthquake damage to wells documented occurred in the 1976 Tangshan, China earthquake. Of the 70,000 wells, about 64 percent of the wells in the strongly shaken area were damaged. Some were silted up with sand (020).

B. TRANSMISSION, DISTRIBUTION AND COLLECTION SYSTEMS

Water transmission and distribution and sewage collection and transport systems are very important parts of any urban area's public works. History indicates that such transport systems are vulnerable to earthquake induced damages. Seismic activity has caused either partial or total disruption of water supply or sewage collection piping, aqueducts and channels in urban areas throughout the world. In some instances, the loss of vital transport systems has resulted in destruction of both lives and property.

Transport systems in this section are categorized as follows:

- major transmission systems - tunnels, large diameter pipelines, covered conduits and open channels
- distribution and collection systems - buried pipelines and appurtenant structures, service laterals and connections to structures

Major transmission systems are categorized separately from distribution and collection systems in this discussion for a number of reasons. Where pipelines are used for transmission, they are often of much larger diameter than those used in distribution and collection and are, therefore, less flexible. Transmission pipelines are sometimes laid above-ground, while distribution and collection systems are usually buried. Transmission systems are particularly crucial as they often transport a single source or one of a few sources of water to the distribution system which is commonly a network where failure of a single line will not be critical. Transmission lines must sometimes traverse long distances and unavoidably cross fault zones as is the case in the Los Angeles and San Francisco areas. Major fault crossings may sometimes be avoided with local distribution systems.

The effects of earthquakes on segments of transmission, distribution and collection facilities can be categorized by failure mode. Damage to these facilities may be caused by seismic induced earth movements, such as surface faulting, tectonic uplift and soil failures (i.e., landslides, liquefaction and compaction of soils). The other major cause of damage is direct seismic shaking, which may induce axial and binding stresses on the structure.

Many engineers have analyzed the failure modes associated with transmission, distribution and collection facilities subjected to earthquakes. Damage reports from previous earthquakes and engineering analyses form the basis of the following survey of potential damage to water and sewage transport systems. Pumping stations will be discussed in Section C of this chapter.

MAJOR TRANSPORTATION SYSTEMS

This section includes a discussion of tunnels, covered conduits, open channels and large diameter pipelines. Potential damages of transmission system fault crossings, surface supported piping, seismic induced lateral earth pressures and rock tunnels are included in this section as they are more closely related to transmission than to distribution facilities. A discussion of seismic shaking, while pertinent to transmission structures, is included in the distribution and collection system subsection. The effects of soil failures, i.e., liquefaction and landslides, are included; however, a more detailed discussion of these phenomena can be found in Chapter 5 of this report.

Transmission systems crossing fault zones may be subject to large differential ground surface movements. Kennedy et al. (043) point out that fault crossings are a great hazard to oil transmission pipelines traversing long distances. Relative vertical and horizontal movements of adjoining geologic blocks can exert compressive, tensile and/or shearing stresses on a transmission structure. The magnitude of these stresses and thus the extent and type of failure depends on the amount and type of relative displacement of the adjoining blocks. The simplified illustration below depicts a shear failure of a buried pipeline at a fault crossing.

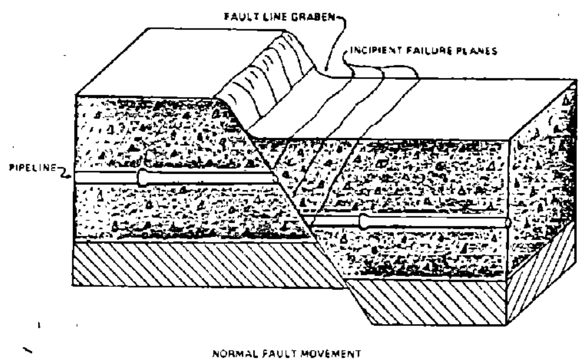


Figure IV-1. Example of pipeline failure at a fault crossing (044).

Seismic shaking may induce axial and bending stresses on transmission structures as well. Transmission facilities may be more vulnerable to bending than distribution piping because of the larger pipe/channel cross sections, reducing the structure's flexibility. (See distribution piping sub-section for a detailed discussion.)

Some basic types of failures caused by the stresses identified above are outlined below:

- crushing and breaking of joints and buckling of channels and pipes due to compression
- pull-out or separation of joints due to tension
- shearing of transmission structures or off-setting of joints
- alteration of gravity flow

- bending or shear failure of open channel and covered conduit walls due to lateral earth pressure

Other variables that determine the type and extent of damage to a transmission structure include:

- the ductility of the construction material
- whether the pipeline structure is above-ground or buried
- depth of burial and backfill material used
- the angle at which the structure crosses the fault

Some water transmission pipelines are constructed above-ground. Unlike buried pipelines which are constrained to respond as the surrounding soil media responds, above-ground pipelines' response to earthquakes depends on the forces induced on the anchor points and the structural parameters of the pipeline, which include:

- distance between anchor points
- the rigidity of the pipeline
- the weight of the pipeline

A report from the oil transmission industry (043) stated that above-ground pipeline failure resulted primarily from support structure failure, attachment to the pipe and movement.

Much of the damage to major water transmission systems during the 1971 San Fernando earthquake occurred in a zone of tectonic ruptures just north of the Upper Van Norman Reservoir. Table IV-2 provides descriptions of damage to transmission pipelines in that area. Four steel pipelines with welded slip joints and one riveted steel pipeline sustained major damage. They ranged in size from 50-96 inches in diameter. Damage to the transmission pipelines were caused both by horizontal and vertical ground displacements

OF WATER AND POWER (LADWP) FOLLOWING THE 1971 SAN FERNANDO, EARTHQUAKE (033)

EARTHQUAKE/ AFFECTED AREA	TYPE OF PIPE	PIPE DIAMETER mm* (in.)	DESCRIPTION OF DAMAGES
San Fernando	Transmission Piping		
Feb. 9, 1971	Steel: Welded slip joints	1270 (50)	All failures occurred at the joint - 2 mechanical couplings and 8 welded slip joints suffered damages. Failures were predominantly compressive but some tensile failures did occur.
Los Angeles, Ca. (Los Angeles Dept. of Water at Power (LADWP)	Steel: Welded slip joints	1370 (54)	5 joints were damaged (one a mechanical coupling in a vault). Failures occurred at the belled section of the joint.
	Steel: Above-ground	1370 (54)	Ring girders which anchor pipeline to concrete pile caps were distorted.
	Steel: Welded slip joints (above-ground)	2440 (76)	4 mechanical couplings and 1 welded slip joint failed due to tension. Also one compressive failure in the pipe body. Support piers and anchor blocks were displaced due to a landslide.
	Riveted steel (above-ground)	2440 (96)	2 expansion joints elongated approximately one foot at each location, shattering anchor blocks. Moderate buckling of pipe body at pier contact and anchor points. Supporting piers were displaced vertically and horizontally by as much as two feet.
	Distribution Piping		
	Steel: Uncoated		Leaks in the form of small blow holes and larger blowouts resulted from the internal water pressure on the walls weakened by corrosion and earthquake movement.
	Cast iron		Circumferential cracks and shattering of pipe bodies.
	Cast iron [1]Bell-and-spigot, cement-caulked joint		Greatest number of joint failures occurred with this type of pipe. Most joint failures were attributable to tensile forces causing pull-out of the joint.
	Cast iron: [2]Bell-and-spigot, lead caulked joint		Both tension and compressive failure at joints. (Compressive failure often resulted in broken bell at the joint).
	Service Connections		
	Copper		Broken corporation valves, ball and socket elbows, and curb valves.
	Galvanized		Broken elbows and couplings. Breaks also occurred in the pipe body.

[1] Considered to be inflexible joint.

[2] Considered to be a somewhat flexible joint.

* Millimeters rounded to nearest 5 mm.

and ground failures (landslides). The majority of the failures occurred at joints. Other types of failure included elongation or buckling of the pipe body and displacement of above-ground pipeline pier supports and ring girder anchors (033).

The 76-inch welded steel pipeline (see Table IV-2) was constructed above ground on a hillside. A landslide displaced anchor piers axially, resulting in pull-out or tensile failure of mechanical couplings and welded slip joints near the summit, and buckling of the pipe body near the mid-slope. A 96-inch riveted steel pipeline was also laid above-ground. Expansion joints and the pipe body were elongated by as much as one foot. Buckling of the pipe body at pier support contacts also occurred. Damages were a direct result of the pier supports being displaced vertically and horizontally by as much as two feet due to tectonic uplift (033).

Damage to major steel trunk lines was also attributed to the combination of seismic shaking and ground movement during the San Fernando earthquake. Failures were the result of the pipe pulling apart at flexible couplings, the coupling dropping down, and the pipe, while attempting to return to its original position, crushing the coupling. The couplings involved were short couplings and used primarily for flexibility. However, they were not designed to withstand axial displacement (045).

Other major transmission facilities included concrete-lined tunnels, open channels and covered conduits. These structures were constructed from both reinforced and unreinforced concrete. The First Los Angeles Aqueduct consists of tunnel reaches lined with unreinforced concrete. The aqueduct, constructed in 1913, measures approximately 10 feet wide by 10½ feet high. Although no severe damages occurred, fractures of the concrete lining, primarily circumferential, ranging from hairline cracks to ¼ inch in width were revealed by inspection (033). Two covered box conduits, the Maclay and Chatsworth High Lines, were damaged during the San Fernando earthquake. Damage to the conduits consisted of several cracks and spalling (033).

A flood control channel, the Wilson Canyon Channel, is a covered rectangular box conduit which crossed a segment of the main fault break of the San Fernando earthquake. Damages included separation of the conduit at or near the joint due to longitudinal bending and spalling, cracking, and bulging of conduit walls and offsetting of joints due to lateral earth pressures. All of the major damage to the conduit was attributed to fault displacement. The offsets on either side of the break were as much as 4, 6.5, and 4 feet vertically, transversely, and longitudinally, respectively (033).

Unreinforced concrete-lined trapezoidal open channels which were part of the Los Angeles aqueduct system were heavily damaged by the San Fernando earthquake. Again, lateral ground movements and surface uplifts due to faulting caused heavy fracturing and displacement of the channel lining.

It is evident that buried transmission structures are very susceptible to damage from fault movements. Transverse fault slippage causes the lateral earth pressure to increase on the side walls of such structures. For a box conduit or an open channel, transverse slippage can mobilize the "passive pressure" of the surrounding soil (the wall pushing against the soil). The increase in lateral loads on the side walls of such structures due to passive pressure is much larger than the "active pressure" such structures are typically designed to resist. For box conduits, mobilization of passive pressure could result in a ten-fold increase in design loads on the structure (046). Seismic shaking may also increase the active lateral earth pressure on the "retaining walls" of transmission structures.

All transmission pipeline damages from the 1906 San Francisco earthquake outside the city occurred on wooden trestle supported pipelines, at fault intersections, and from unequal ground settlement. Of three transmission lines feeding the city, only the San Andreas line, a 36-inch riveted iron pipe, was in service within two weeks following the earthquake. This line suffered only one break at a flexible joint. The 30-inch Pilarcitos line was located adjacent to the fault crossing it in a number of locations for a distance of six miles. The fault moved six to seven feet shattering, telescoping and tearing apart the line which was ultimately abandoned (047).

Dowding (048) conducted an analysis of damages to rock tunnels caused by seismic shaking in California, Alaska, and Japan. The case studies included 71 water and railway tunnels. The study correlated surface accelerations measured at the site of the tunnel and incidences of damage. No damage was observed below calculated surface accelerations of 0.2 g, and only minor damages (cracking) was observed below surface accelerations of 0.5 g. Based on the correlations presented in the study, it would appear that rock tunnels are capable of withstanding moderate accelerations, without sustaining major failure. The study also indicated that concrete lined and pressure grouted tunnels are less susceptible to cracking and rock falls than unlined tunnels.

DISTRIBUTION AND COLLECTION SYSTEMS

Numerous accounts of damages to water distribution and sewage collection pipelines have been reported from previous earthquakes. Post-earthquake surveys indicate three major causes of pipeline damage:

- large displacements (pipes crossing fault planes or pipes located in areas of surface fracturing)
- ground failure (i.e., landslides, liquefaction, etc.)
- seismic shaking of pipes

Pipe failure modes caused by fault displacement and surface fracturing are "straight forward". Designs to resist failure require a knowledge of fault locations. If they are known, flexibility can be constructed into the piping system. Soil failure can be predicted based on various soil parameters but prevention can be very costly. The direction and magnitude of movement after failure would, however, be difficult to predict. Therefore, there has been little emphasis put on earthquake induced pipeline failure analysis from these potential modes. On the other hand, pipeline seismic shaking allows a "straight forward" theoretical analysis. The large majority of seismic resistant pipeline design analysis has been done in this area.

The following discussion will give descriptions of the types and causes of failure sustained by distribution and collection systems subjected to past earthquakes. Quantitative and qualitative analyses of damages and failure modes will also be referred to where appropriate.

Seismic Shaking of Buried Pipelines

Response to the seismic free field, shaking or vibration of buried pipelines is one of the main causes of failure of these structures. "The seismic free field is the definition of the ground motion, without regard to its modification due to the structure to be analyzed" (049). As indicated in Chapter II, there are three main groups of waves that are transmitted through the earth from an earthquake epicenter. These are known as primary (P), secondary (S) and surface (L) waves. Primary and secondary waves are chiefly responsible for direct seismic shaking of buried pipelines.

Since these waves are transmitted primary in a radial direction from the epicenter, the orientation of the pipeline with respect to the epicenter is an important factor in determining forces exerted on a pipeline, as shown in Figure IV-2.

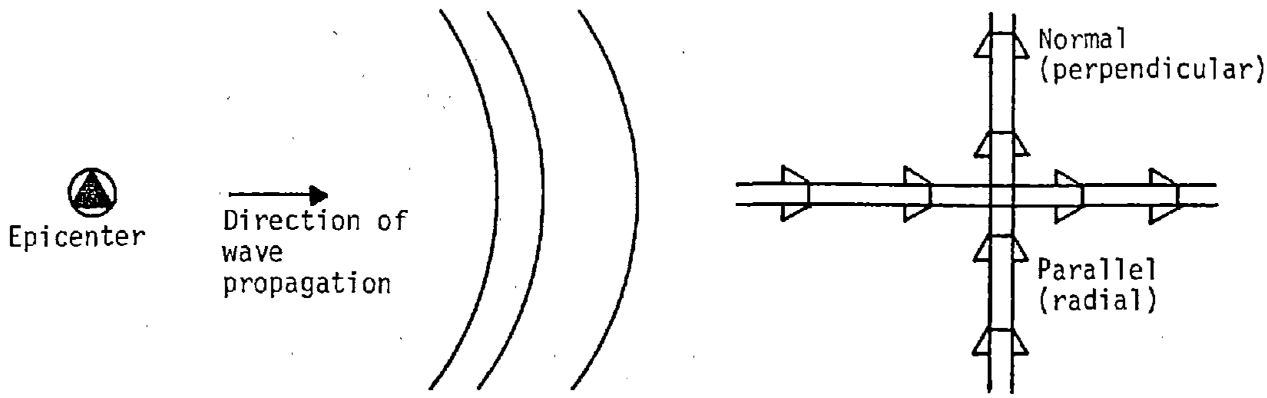


Figure IV-2. Simplified illustration of pipelines parallel and normal to direction of wave propagation (050).

Kubo investigated the relation between damage intensity and direction of water pipelines relative to the direction of wave propagation (051). Evidence of damage to pipelines from the Kanto (1923), Fukui (1948), and San Fernando (1971) earthquakes indicate that parallel or radial oriented pipelines with respect to the direction of the travelling seismic wave are more severely damaged than those with a perpendicular orientation. Data from the San Fernando earthquake indicates that pipelines with a north-south orientation had 3.58 failures/km, while pipelines with an east-west orientation 1.48 failures/km. The location of the epicenter was about 8.7 miles (14 km) north-northeast of San Fernando (052).

Newmark (001), Wang (050) and others have studied the types of waves generated by earth movements at the epicenter of an earthquake. Longitudinal (pressure) waves are created by horizontal earth movement which tend to compress and extend the ground. Vertical ground movements at the epicenter (or the fault) produce lateral and vertical shear waves, which tend to exert bending strains in the ground. Figure IV-3 shows the types of waves generated by horizontal and vertical fault movements, and the corresponding strains exerted upon a buried pipeline parallel to the direction of wave propagation.

Newmark (053) has developed relationships which describe the axial and bending (curvature) strains exerted on the soil surrounding a pipeline. The strain, ε, in the ground in an axial direction is directly proportional to the maximum velocity of the ground movement at the point in question, V, and inversely proportional to the velocity of the pressure wave propagation through the ground, C_p, as expressed below:

$$\varepsilon \text{ (axial strain)} = V/C_p$$

Likewise, a relationship for the bending or curvature (χ) can be expressed by:

$$\chi \text{ (curvature)} = A/C^2{}_s$$

where A is the maximum acceleration at the point in question and C_s is the velocity of the shear wave moving through the ground, relative to the pipeline (053).

The relationships for determining the axial strain and curvature of the ground can be used to determine the corresponding strain and curvature on a pipeline, providing two assumptions are made. The first assumption is that the relative motion between the pipe and surrounding soil is negligible as the soil damping is relatively high and the frequencies of earthquake ground motion are lower than the natural frequency of a buried pipeline. The second assumption is that the shapes of the seismic waves remain constant as they move through the ground. The only difference in the effects of the wave on two different points of a pipeline will be governed by a time lag, a function of separation distance and wave speed. However, this requires that

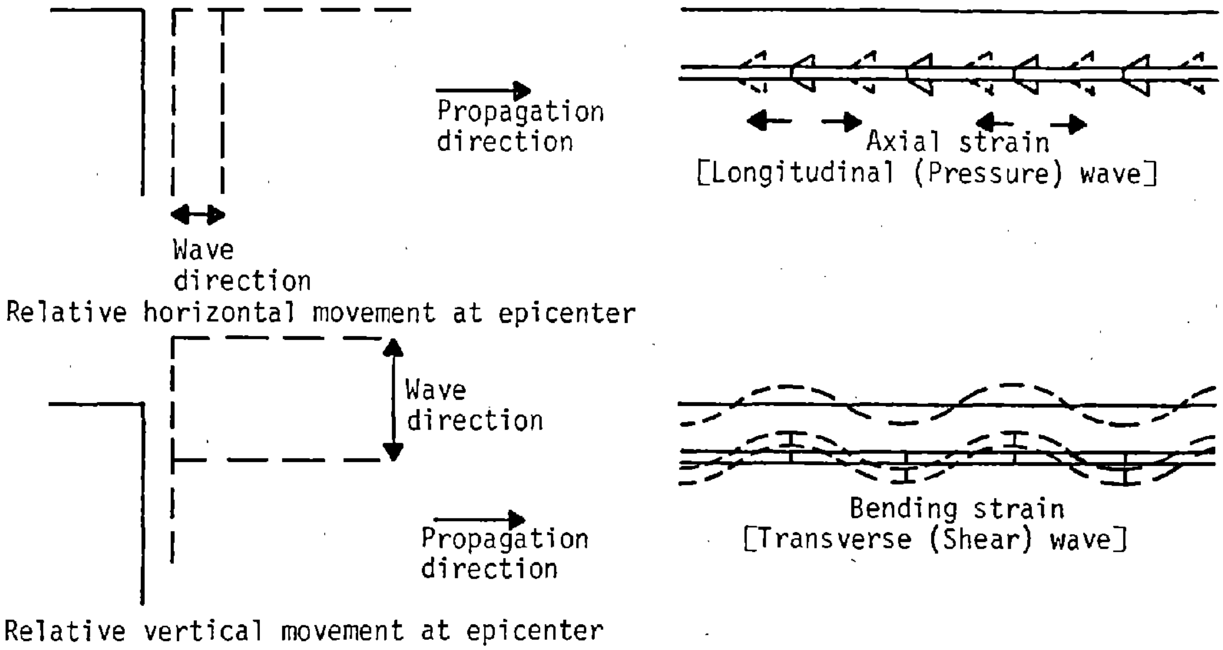


Figure IV-3. Simplified schematic illustration of bending and axial strain due to earthquake induced ground movement. (Pipe parallel to direction of wave propagation) (050).

the seismic wave travel through a homogeneous soil medium. Providing these assumptions are correct, axial strains exerted on the pipeline will be equal to the axial ground strains (054,055,056,050).

With these two assumptions, the axial strain in the soil will be the same as the axial strain on the pipeline, providing there is no relative displacement at the joints of the pipeline. If the pipeline lengths are taken to be rigid with the midpoint of each responding exactly with the surrounding soil, then the soil strain will be taken up in the pipe by the relative pipe joint displacement, both axial and rotational (057). Figure IV-4 illustrates schematically axial joint displacement and joint rotation of a segmented pipeline.

Wang (057), utilizing the two previously-stated relationships in a simplified analysis approach for axial strain (ε) and curvature (χ), has stated the following relationships for the upper bounds of maximum joint displacement, $U_{p\ max}$, and maximum joint rotation, $\theta_{p\ max}$

$$U_{p,\ max} = \varepsilon_{max}\ L = V_{max}\ L/C_p$$

$$\theta_{p,\ max} = \chi_{max}\ L = A_{max}\ L/C_s^2$$

where V_{max} is the maximum ground velocity and A_{max} is the maximum ground acceleration; C_p and C_s are the longitudinal (compressive) and transverse (shear) wave propagation velocities in the surrounding soil, respectively; and L is the length of the pipe segment.

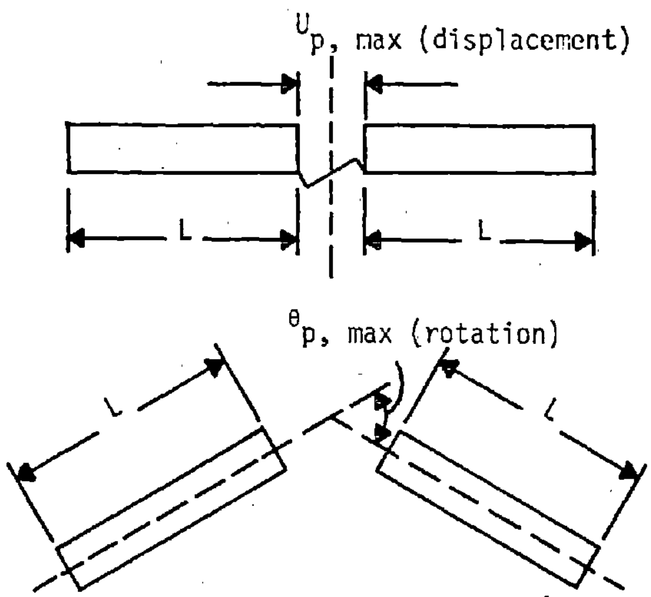


Figure IV-4. Maximum joint displacement and rotation of a segmented pipeline (057).

Wang (058) has taken the pipeline analysis one step further by developing a quasi-static analysis. Some of the parameters included in this computer analysis are pipe length, material, pipe joint stiffness and soil stiffness as well as earthquake input.

It is currently difficult to apply these pipe response analyses to design due to a lack of understanding of earthquake wave forms, lack of detailed information on soil response parameters and lack of empirical results from material testing. Some information is available in the literature on various parameters (059,060,061,062).

Wang (050) has reported on the application of conservative pipeline system (18" pipeline) parameters to his quasi-static analysis procedure. This analysis indicates that bending stresses in a continuous pipe (the worst case) constitute approximately two percent of axial stresses and are, therefore, not significant. Axial stresses in continuous welded steel pipe are the only ones that are greater than (only slightly greater than) the pipes' yield strength. Wang also pointed out that experience has shown that this maximum strain condition would probably not occur due to the invalidity of the first assumption, soil/pipe relative movement at this extreme. No earthquake induced stresses were significant for jointed pipelines. The maximum strain reported was approximately 8×10^{-2} (continuous pipe). The maximum joint displacement for a 20' pipe segment was approximately 1/3 inch. Weidlinger (063) has reported rough estimates of axial and bending strains that are not inconsistent with Wang's.

As one might expect from these results, pipeline damage from earthquake shaking is primarily associated with axial pipe failure, not bending.

Joint failures due to axial displacement include pull-out or separation of the joint due to tension and cracking or deflection of the joint due to compression. Pipe joint rotation can cause failure of the joint in flexure, especially on large diameter pipes whose joints will not permit as much rotation as smaller diameter pipes.

In particular, the 1923 Kanto earthquake damaged many water pipelines due to direct seismic shaking, which resulted in pipeline breaks and separation and loosening of joints (Table IV-3). Most of the damage to the water pipeline network in Managua, Nicaragua from the 1972 earthquake consisted of pull-out of joints, loosening of bell-and-spigot joints and joint gasket displacement due to longitudinal deformation (034).

The San Fernando earthquake also caused joint failure of water and sewer pipes to occur through a number of failure modes including seismic shaking. Joint failures included pull-out, crushing or splitting of the belled portion of bell-and-spigot joints and joint misalignment, caused by tensile, compressive and lateral forces, respectively. Joints were damaged in a wide variety of pipes, including concrete, vitrified clay, steel, riveted steel and cast iron. Table IV-4 indicates the percentage of the type of water pipe with associated type of joint which had to be replaced following the earthquake (033).

Table IV-5 indicates damages to the sewer systems of San Fernando and Los Angeles, California which were caused by the San Fernando earthquake. The percentage of sewer pipe which had to be reconstructed following the earthquake is categorized by type of joint for the pipeline or whether the pipeline was encased. All sewer pipes were of vitrified clay material. Sewer pipes with flexible joints (gasket-type) sustained less damage than sewer

TABLE IV-3

DAMAGES TO WATER SYSTEM OF TOKYO FOLLOWING THE 1923 KANTO EARTHQUAKE (005)

EARTHQUAKE/ AFFECTED AREA	TYPE OF PIPE	PIPE DIAMETER mm* (in.)	DESCRIPTION OF DAMAGES: BREAKS AND JOINT SEPARATIONS**	BREAKS OF HYDRANTS**	BREAKS OF VALVES**
Kanto Sept. 1, 1923 Tokyo, Japan	Cast Iron	75-400 (3-16)	214	219	109
	Cast Iron	400-1100 (16-43)	10		

EARTHQUAKE/ AFFECTED AREA	TYPE OF PIPE	PIPE DIAMETER mm* (in.)	JOINT LOOSENINGS† No./km (No./mi.) MAJOR	MINOR
	Cast Iron	75-125 (3-5)	134 (216)	152 (245)
	↓	150 (6)	129 (208)	155 (250)
	↓	200 (8)	131 (211)	146 (235)
	↓	250 (10)	113 (182)	102 (164)
	↓	300-350 (12-14)	106 (171)	109 (175)
	↓	400-1100 (16-43)	87 (140)	102 (164)

* Millimeters are rounded to nearest 5 mm.

** Reported as of Jan. 31, 1924.

† Number of Joints which were recaulked before May 31, 1924.

TABLE IV-4

DAMAGES TO THE WATER DISTRIBUTION SYSTEM OF SAN FERNANDO, CA. FOLLOWING THE 1971 SAN FERNANDO EARTHQUAKE (033)

EARTHQUAKE/ AFFECTED AREA	TYPE OF PIPE	PIPE DIAMETER mm* (in.)	DESCRIPTION OF DAMAGES		
			TOTAL LINEAR METERS (FEET)**	LINEAR METERS (FEET) OF PIPE REPLACED**	PERCENT OF PIPE REPLACED
San Fernando	[1]Cast iron				
Feb. 9, 1971	Bell-and-spigot	100 (4)	3,180 (10,450)	-- --	
San Fernando, Ca.	Bell-and-spigot-mineral lead, cement, and rubber gland joints	150 (6)	44,420 (145,789)	3,670 (12,037)	7
	Bell-and-spigot-cement caulked and rubber gland joint	200 (8)	14,810 (48,620)	690 (2,265)	
	Bell-and-spigot	350 (14)	1,350 (4,431)	-- --	
	[1]Thin-walled riveted steel:				
	Bell-and-spigot	150 (6)	1,480 (4,870)	680 (2,235)	
	Bell-and-spigot	200 (8)	2,290 (7,530)	220 (720)	22†
	Bell-and-spigot	250 (10)	2,060 (6,779)	402 (1,320)	
	Bell-and-spigot and riveted joints	300 (12)	--	440 (1,460)	
	Bell-and-spigot and riveted joints	350 (14)	--	1,400 (4,585)	
	Concrete steel cylinder-bell-and-spigot, rubber gland joint	450 (18)	1,670 (5,493)	370 (1,200)	22
	Standard steel casing (screw joint coupling)	50-100 (2-4)	5,420 (17,790)	--	
		150 (6)	7,650 (25,115)	260 (856)	2
		200 (8)	2,520 (8,260)	--	
		250 (10)	790 (2,600)	--	

[1] Categorized by type of joint(s).

* Millimeters are rounded to nearest 5 mm.

** Length in meters is rounded to nearest 10 meters.

† This percentage is based on 6, 8, and 10 inch diameter pipe, since original length of 12 and 14 inch diameter pipe was not reported.

TABLE IV-5

DAMAGE TO SEWER SYSTEMS OF LOS ANGELES AND SAN FERNANDO FOLLOWING THE 1971 SAN FERNANDO EARTHQUAKE (033)

EARTHQUAKE/ AFFECTED AREA	TYPE OF PIPE	PIPE DIAMETER mm* (in.)	DESCRIPTION OF DAMAGES			
			LENGTH OF SEWER PRIOR TO EARTHQUAKE METERS** (FEET)	LENGTH TO BE RECONSTRUCTED METERS** (FEET)	PERCENT TO BE RECONSTRUCTED PER JOINT	PERCENT TO BE RECONSTRUCTED TOTAL
San Fernando Feb. 9, 1971 San Fernando and Los Angeles, Ca.	Sewers[1] Vitrified clay					
	Flex[2]	200 (8)	110,780 (363,600)	17,790 (58,400)	16.1	
	Rigid[3]	200 (8)	27,600 (90,600)	9,410 (30,900)	34.1	19.7
	Encased[4]	200 (8)	370 (1,200)	80 (260)	21.6	
	Flex	250 (10)	7,250 (23,800)	1,220 (4,000)	16.8	
	Rigid	250 (10)	1,610 (5,300)	300 (1,000)	18.6	17.2
	Encased	250 (10)	80 (270)	10 (40)	14.8	
	Flex	300 (12)	7,590 (24,900)	1,190 (3,900)	15.7	
	Rigid	300 (12)	2,960 (9,700)	850 (2,800)	28.8	22.4
	Encased	300 (12)	1,040 (3,400)	550 (1,800)	52.9	
	Flex	375 (15)	4,230 (13,900)	1,070 (3,500)	25.2	
	Rigid	375 (15)	1,070 (3,500)	550 (1,800)	51.4	20.2
	Encased	375 (15)	1,100 (3,600)	300 (1,000)	27.8	
	Flex	450 (18)	5,420 (17,800)	1,490 (4,900)	27.5	
	Rigid	450 (18)	2,440 (8,000)	1,860 (6,100)	76.2	30.0
	Encased	450 (18)	2,220 (7,300)	1,710 (5,600)	76.7	

1 Categorized by type of joint.
2 Flex - Flexible bell-and-spigot joint, with PVC or polyurethane compression ring.
3 Rigid - Rigid bell-and-spigot joint, with cement mortar packed caulking.
4 Encased - Partial or complete concrete encasement of sewer.
* Millimeters are rounded to nearest 5 mm.
** Length in meters is rounded to nearest 10 meters.

pipes with rigid joints (mortared, etc.). In larger diameter pipes, in particular 15 and 18 inch (375 and 450 mm) diameter pipe, flexible joints were of increasing importance in reducing damages (064).

Katayama, Kubo and Satu (065,005, 066) have investigated the relationship between maximum ground accelerations and seismic shaking induced failure of water pipelines. Figure IV-5 plots failure ratios of water pipelines of past earthquakes, versus estimates of the maximum ground accelerations reported for the affected geographic area. Of these earthquakes, the damages to the water distribution pipes following the San Fernando earthquake were studied in detail (065). LADWP's system was divided in grids with the ground acceleration estimated in each (decreases as one moves away from epicenter). A failure ratio from seismic shaking (excludes failures from faulting) was calculated for each section with the results plotted in Figure IV-5.

The relationship plotted may be expected after reviewing Newmark's relationship for pipe axial strain:

$$\varepsilon_{max} = \frac{V_{max}}{C_p}$$

V_{max} is directly proportional to the maximum ground acceleration as reported by Wang (059).

The authors noted the limitation of maximum ground accelerations as a suitable measure to describe the intensity of ground shaking that is related to the failure of buried pipelines. However, the relationship shown in Figure IV-5 is a reasonable first approximation of the extent of damage one might expect.

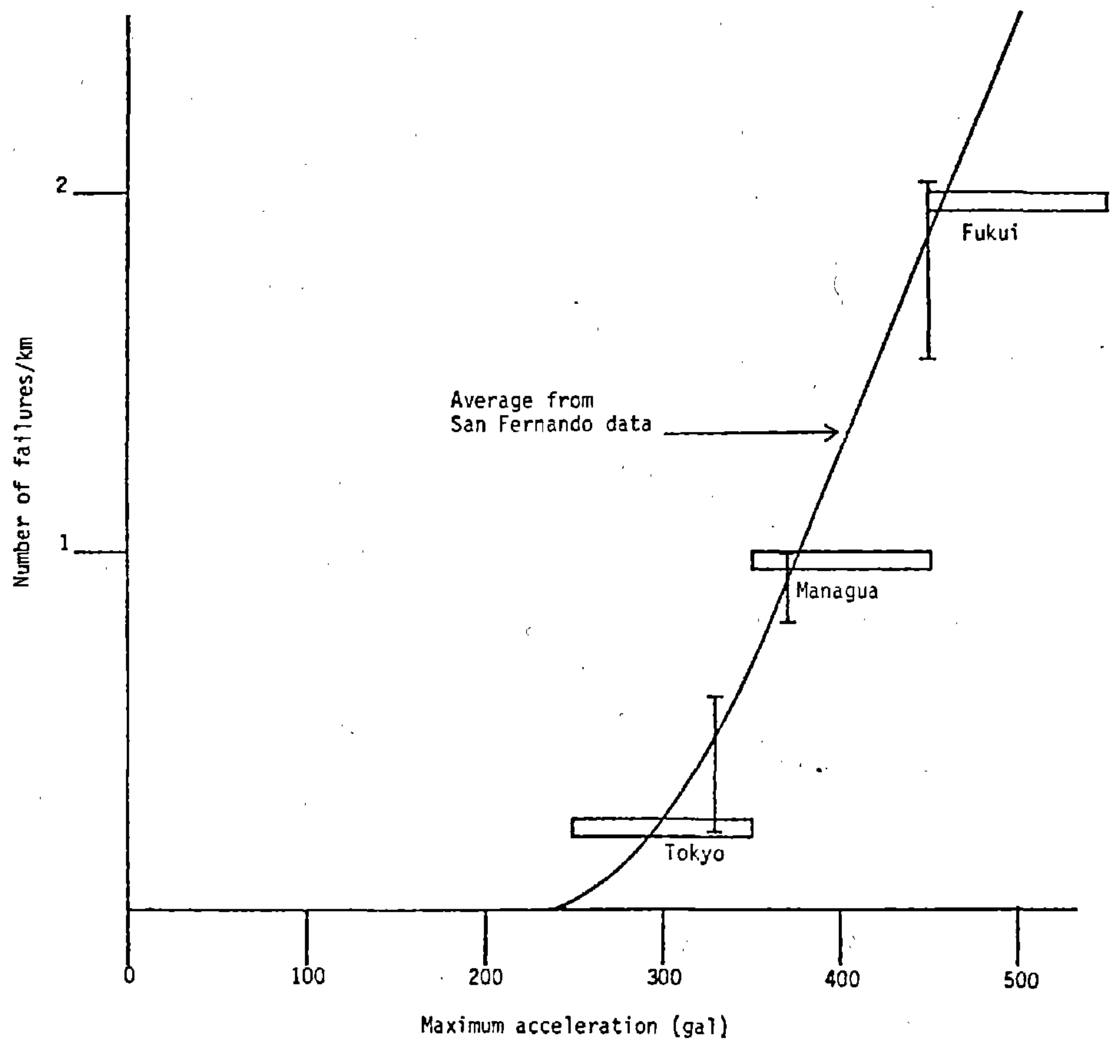


Figure IV-5. Failure ratio of water pipes and ground shaking intensity (065).

Seismic Shaking and Soil Variability

Studies of both experimental and observed data on damage incidence to buried pipelines during an earthquake and the relation of such damages to the surrounding soil properties have been conducted (065, 067, 051, 068, 069, 070, 071, 072).

Soil stiffness has a compounded effect on pipe response. Wang has reported that the ratio of V_{max}/A_{max} increases as the ground becomes softer (059). In addition, the propagation velocity of axial waves, C_p, decreases as the ground stiffness decreases. Therefore, in Newmark's equation for axial strain, with A_{max} constant, we have:

$$\text{greatly increases} \leftarrow \varepsilon_{max} = \frac{V_{max}}{C_p} \begin{array}{l} \rightarrow \text{increases} \\ \rightarrow \text{decreases} \end{array}$$

Wang (057) has reported a conservative approximation on wave propagation velocity as a function of the soil shear modulus and soil mass density. He also relates the soil resistance spring constant used in the quasi-static analysis to the soil shear modulus.

By studying failure ratios of pipeline damage from past earthquakes, Kachadoorian concluded that earthquake damage to pipelines would be least in bedrock, intermediate in coarse-grained sediment, and maximum in fine-grained sediment (073).

The buried pipeline often passes through different horizontally adjacent soil layers with varying soil properties also defined as soil transition zones. Waves passing through these transition zones may not follow the assumption that the seismic wave shape will remain constant as they propagate through the ground.

Nelson and Weidlinger (074) have identified three ways in which the local soil inhomogeneity may affect the axial response of the buried pipeline:

- increased phase delay - the effect of which is less than the phase delay in soft soils
- pipe joint response amplification (by as much as 2) due to the change in the waveform passing through the inhomogeneity
- change in soil stiffness will cause joint relative motion on order of magnitude larger than that caused by phase delay.

Hindy and Novak (055) investigated the soil-pipe interaction in soil composed of two different media separated by a vertical boundary. Ignoring any relative motion between the pipe and soil, they concluded that pipe stresses were highest near the boundary of the two horizontally adjacent soil media.

The effects of different soil media in relation to seismic shaking and damage incidence of pipeline failures have been investigated by Katayama, Kubo, et al. using statistics from the 1923 Kanto earthquake (065, 005, 068). The metropolitan area of Tokyo was divided into a grid system consisting of 96, one kilometer square subdivisions. Each mesh within the grid was classified by soil characteristics, the relevant response frequency of the mesh, and an average damage index (failures/km).

The meshes where the heaviest damage occurred were located in a region between firm ground at the hillside and soft ground in the downtown district. The effects of the ground shaking in the two regions differed considerably, with the ground motion being severe in the soft alluvium and less severe in the firm loam ground. The differential relative response between the adjacent soil layers caused axial deformation of the pipe.

Damage to the water distribution network of Sendai, Japan due to the 1978 Miyagiken-Oki earthquake offers more evidence of seismic shaking induced pipeline failure as a result of inhomogeneous soil conditions in combination with soil failures. Most of the damage to pipelines was concentrated in areas of newly-developed residential areas. These areas were developed by terracing the surrounding hillsides of Sendai with large-scale cut-and-fill operations, where strong seismic shaking caused fissures, settling, slippage and relative displacements, due to the instability of artificial slopes, inadequate consolidation of fills and abrupt changes in subsoil properties between cut and fill (022). Table IV-6 indicates failure ratios of pipelines damaged during the 1978 Sendai earthquake.

Weidlinger (049) has concluded that soil transition zones do not by themselves significantly effect the response of pipelines. In addition, based on the relation of theoretically calculated pipe strains and those that pipe materials can withstand, earthquake induced pipeline failure "cannot be explained by a simple mechanism of ground strain transmission". Weidlinger also observed that in continuous pipelines, there may not be adequate frictional forces (without adhesion of the soil to the pipe) between the pipe and surrounding soil to prevent amplification of the ground motion in the pipe. With regard to jointed pipelines, there may exist some significant frequency ranges where soil damping cannot occur, and amplification or deamplification of the soil motion may occur.

Wojcik (075) has shown that soil dipping layers may crease resonance zones on the ground surface which amplify translation motion 6 to 15 times the input motion. A dipping layer, shown in Figure IV-6, is a wedge-shaped layer of soil overlying another layer of soil, a geologic formation not uncommon in valleys partially filled with alluvial material.

TABLE IV-6

DAMAGES TO THE WATER SYSTEM OF SENDAI, JAPAN FOLLOWING THE 1978 MIYAGIKEN-OKI EARTHQUAKE (022)

EARTHQUAKE/ AFFECTED AREA	TYPE OF PIPE	PIPE DIAMETER mm§§ (in.)	DESCRIPTION OF DAMAGES: PIPE BREAKAGE														
			CAST-IRON*			STEEL			ASBESTOS-CEMENT			PVC			TOTAL		
			L** km (mi.)	BREAKS	No./km (#/mi.)	L km (mi.)	BREAKS	No./km (#/mi.)	L km (mi.)	BREAKS	No./km (#/mi.)	L km (mi.)	BREAKS	No./km (#/mi.)	L km (mi.)	BREAKS	No./km (#/mi.)
Miyagiken-Oki June 12, 1978/ Sendai, Japan	See Description of Damages	50 (2)	--	--	--	--	39	--	--	30	--	--	28	--	--	97	--
		75 (3)	19 (12)	3	0.16 (0.25)	1 (1)	1	1 (1)	7 (4)	32	4.57 (8)	114 (71)	21	0.18 (0.30)	141 (88)	57	0.40 (0.65)
		100 (4)	184 (114)	15	0.08 (0.13)	2 (1)	0	0 (0)	29 (18)	5	0.17 (0.28)	207 (128)	22	0.11 (0.17)	422 (261)	43	0.10 (0.16)
		150 (6)	205 (127)	4	0.02 (0.03)	1 (1)	0	0 (0)	7 (4)	0	0 (0)	--	--	--	(213) (132)	4	0.02 (0.03)
		200 (8)	119 (74)	2	0.02 (0.03)	1 (1)	0	0 (0)	2 (1)	1	0.5 (1)	--	--	--	122 (76)	3	0.02 (0.04)
		250 (10)	40 (25)	3	0.08 (0.12)	1 (1)	0	0 (0)	1 (1)	0	0 (0)	--	--	--	42 (27)	3	0.07 (0.11)
		300 (12)	70 (43)	2	0.03 (0.05)	4 (2)	0	0 (0)	2 (1)	1	(0.5) (1)	--	--	--	76 (46)	4††	0.05 (0.09)
		350 (14)	2 (1)	0	0 (0)	--	--	--	--	--	--	--	--	--	2 (1)	0	0 (0)
		400 (16)	28 (17)	2	0.07 (0.12)	4 (2)	0	0 (0)	--	--	--	--	--	--	32 (19)	2	0.06 (0.10)
		450 (18)	2 (1)	0	0 (0)	--	--	--	--	--	--	--	--	--	2 (1)	0	0 (0)
		500 (20)	20 (12)	1	0.05 (0.08)	7 (4)	0	0 (0)	--	--	--	--	--	--	27 (16)	1	0.04 (0.06)
		500-1100 (20-43)	16 (10)	0	0 (0)	50 (31)	0	0 (0)	--	--	--	--	--	--	66 (41)	0	0 (0)
		TOTAL	705 (436)	32	0.04 (0.07)	71 (44)	1§	0.01 (0.02)	48 (29)	39§	0.81 (1.34)	321 (199)	43§	0.13 (0.22)	1145 (708)	117§	0.10 (0.17)

* [illegible]

** [illegible]

† Including one break of an isolating valve.

†† Including one break of a hydrant.

§ Does not include damages to 50 mm (2 in.) pipe due to lack of information.

§§ Millimeters are rounded to nearest 5 mm.

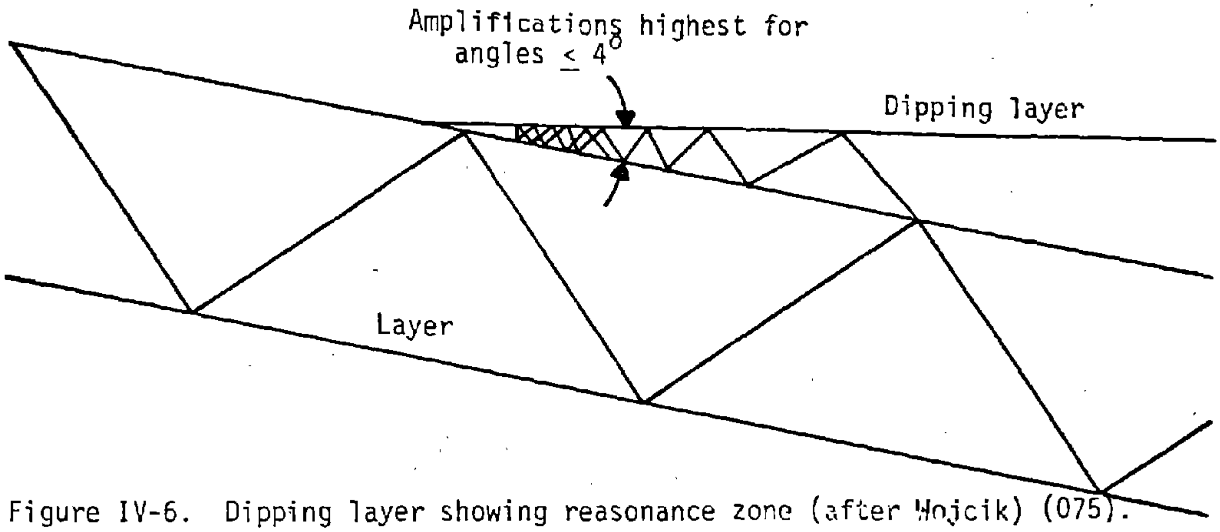


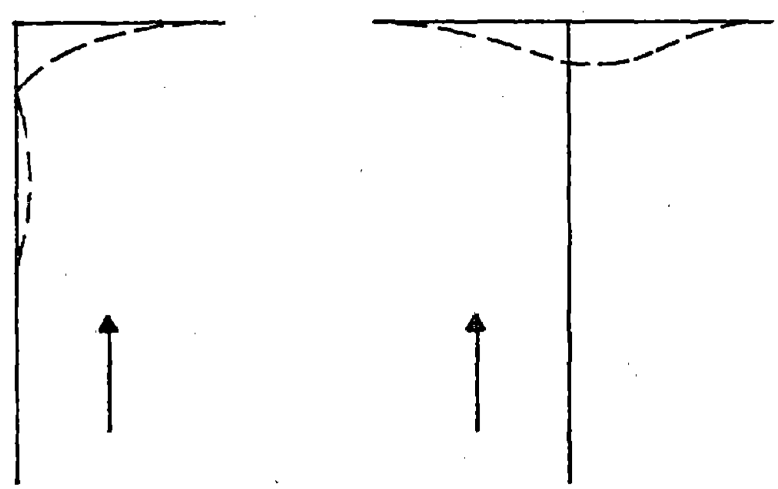
Figure IV-6. Dipping layer showing reasonance zone (after Wojcik) (075).

The work on dipping layers was induced by the observation that water lines near the southern edge of the San Fernando Valley were damaged in the 1971 earthquake more than similar lines in the middle of the Valley that were closer to the epicenter. A similar observation was made by Poceski (1969) of the 1963 Skopje earthquake (075).

Seismic Shaking and Pipeline Appurtenances

Tee junctions, valves, connections to structures, service laterals and hydrants are examples of pipeline appurtenances. Appurtenances represent discontinuities in the pipeline's structural system. Salvadori and Singhal (060) presented the results of previous studies indicating possible stress concentrations in connections and branches 10 to 12 times those found in the pipe under non-seismic conditions.

When these discontinuities are subjected to earthquake motion, the stresses to which they are subjected may be greater than those in a straight pipe for several reasons. If a pipeline is attached to a structure, the structure may have a natural frequency independent of the pipeline's, resulting in out of phase vibrations. If the no-slip assumption made in the pipeline model is correct, the pipelines, no matter what their orientation, should move with the soil with no differential response at pipe tees, elbows and thrust blocks. However, if the pipeline in fact moves with respect to the surrounding soil as suggested by a number of major lifeline researchers (070, 049, 050), pipeline discontinuities may resist this slippage, inducing local stress increases. Shinozuka and Koike (070) and others (076) have proposed methods of calculating these increased pipeline stresses. The point at which slippage may occur (free-field strain) is, however, unclear and requires further investigation. Figures IV-7(a) and IV-7(b) illustrate schematically the pipe deformations on bend and tee junctions respectively, by a pressure wave with a parallel orientation with respect to a segment of the pipe involved in the junction.



IV-7(a) IV-7(b)

Figure IV-7(a) Deformations to right angle and 7(b) tee junctions, respectively, are represented by dotted lines (after Shinozuka and Koike (070).

Fire hydrant laterals, water and sewer house connections, and other points where a pipeline forms a tee or cross intersection or branch are susceptable to seismic shaking induced damages. Damages to house connections due to seismic shaking have been reported for every earthquake included for discussion in this report. Following the 1978 Miyagiken-Oki earthquake, for example, approximately 2000 house connections were broken.

Water service connections are typically of either lead, galvanized iron, copper, or in some instances steel material. Newer construction materials for service connections include polyvinyl chloride and polyethylene. Figure IV-8 below provides an illustration of a typical 1-inch service connection used in the Los Angeles, California service area.

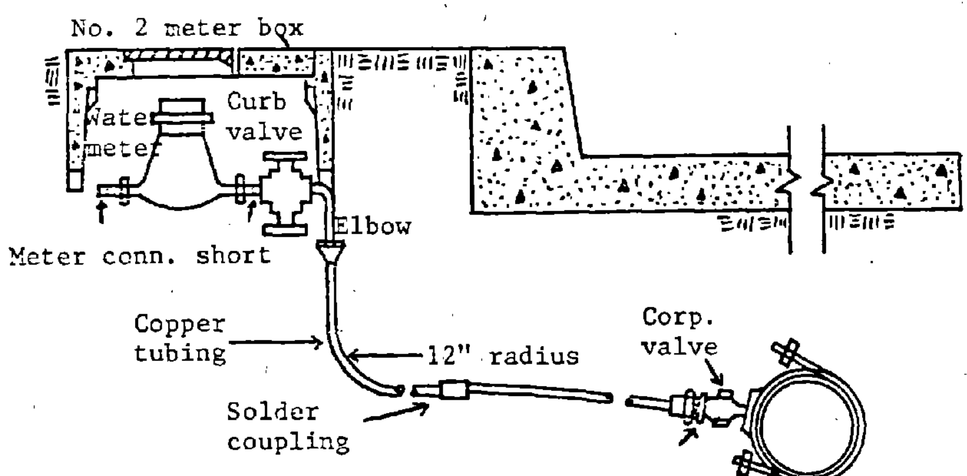


Figure IV-8. Typical 1-inch service connection (033).

The most common failure modes of water service connections are broken corporation valves (cocks), ball-and-socket elbows, and curb valves. Such damages result from the differential relative response of the service connection and the main (which are typically perpendicular) to seismic shaking.

Pull-out of the corporation valve can result in splintering of the main where the fitting was inserted. Other types of damages include sheared couplings between the meter and curb valve and broken service pipes (077).

Valves and hydrants can also be damaged due to seismic shaking. The 1948 Fukui earthquake damaged 152 valves and hydrants (005). During the 1971 San Fernando earthquake, compression of the pipe body into gate valves on water mains broke the belled sections of valves. In some instances, where compression of the main into the valve was severe, the valve was actually split in half as the two connecting pipes were pushed together (033). During the 1923 Kanto earthquake, a total of 109 valves were broken; however, the direct cause of damage was not reported (005).

Hydrants and attached piping were severely damaged during the Kanto earthquake. A total of 219 hydrants were broken, many of them located in an area where fire broke out following the earthquake. This, in addition to several broken mains, crippled the fire fighting potential of metropolitan Tokyo. Consequentially, 44 percent of the downtown area was destroyed by fire.

Attached piping to structures such as storage tanks, wells, pumps, equipment, etc. often fails at the connection of the piping to the structure. Damage to pipe connections is usually a result of either differential relative displacement of the pipe and structure because of ground failure surrounding the structure, differential relative response of the pipe and structure to seismic shaking, or both. The former failure mode will be discussed in the subsection discussing ground failure and potential damage to pipelines.

In past earthquakes, many of the failures to piping attached to structures occurred as a result of ground failure. However, there are some instances such as during the 1964 Alaska (078) and San Fernando (033) earthquakes that attached piping (inlet/outlet) to water storage tanks were broken at the connection due to seismic shaking. Above-ground liquid storage tanks, when subjected to vertical and horizontal ground accelerations, can rock due to sloshing of the tank contents. Thus, strains are exerted on the rigid fitting connecting the piping to the storage tank, causing failure. Consequently, the tank contents may be drained entirely, reducing quantities of stored water for emergency utilization, and possibly resulting in a public safety problem.

Outlet piping attached to well-casings or pumps can be broken at the connection fitting due to the differential relative response of the pipe and the well to seismic shaking. Several wells experienced such damage during the Alaska (078) and San Fernando (033) earthquake. Failures primarily occur at the connection because the well casing or pump and the pipe can resist greater strains than the fitting.

Piping attachments to structures and other types of equipment are discussed in greater detail in Section C of this chapter on potential damage to treatment plants.

Seismic Shaking and Pipeline Corrosion

A pipeline weakened by corrosion is susceptible to damage when subjected to seismic shaking. Corrosion has been known to adversely affect the seismic performance of steel and galvanized steel pipelines and is suspected to affect cast iron pipelines in a similar manner (033, 015, 079). Shaking or pressure surges due to seismic wave propagation can cause corrosion-weakened reaches of pipe to form leaks and/or larger blowouts. Some of the causes of corrosion are the contact of two dissimilar metals with water or soil, stray electric currents, impurities and strains in metals, contact between acids and metals, bacteria in water, or soil-producing compounds that react with metals.

Isenberg (080) has investigated the effect of corrosion on the seismic performance of buried pipelines in three U.S. earthquakes - 1971 San Fernando earthquake, 1969 Santa Rosa earthquake, and the 1965 Puget Sound earthquake, (033, 015, 079, damage reports, respectively). The most prevalent of damage to uncoated steel water mains in the Los Angeles Department of Water and Power (LADWP) system during the San Fernando earthquake was in the form of small holes which resulted from the internal water pressure causing blowouts on sections of pipe weakened by corrosion and subjected to seismic shaking (033). Some 51 leaks in water mains located in the Southeast San Fernando Valley were a result of corrosion weakened pipes being subjected to seismic shaking. Following the earthquake, a 3.3-mile reach of 6-inch steel pipe underlying Ventura Boulevard was examined for leaks. Ten leaks were found in portions of the pipe which were weakened by corrosion. The leak frequency of the pipeline under normal conditions was extremely high, 20 to 30 times above the average of the system. Although ground accelerations and resulting strains on the pipeline could be calculated for the area, the amount of strain needed to cause damage to corrosion weakened pipe was unknown. However, the strains in the area due to seismic shaking were not thought large enough to cause damage to new steel pipe (080).

As a result of the 1969 Santa Rosa earthquake, steel and galvanized steel mains which leaked following the earthquake did so primarily at points which were weakened by corrosion, having a leak frequency ten times the average leak frequency under normal conditions for the system. However, data were not available to determine how many areas with high leak rates under normal conditions did not experience increased leakage as a result of seismic shaking. Therefore, no conclusions for predicting leaks due to seismic shaking from normal leak rates could be drawn. Table IV-7 on the following page indicates damages to water mains as a result of the Santa Rosa earthquake (015).

Of the pipe failures recorded after the 1965 Puget Sound earthquake, approximately 60 percent were attributable to leaks which developed in corroded steel and galvanized steel pipelines subjected to seismic shaking. Typical damage occurred at rust spots or as a result of circumferential splitting of corrosion-weakened threaded connections (080).

The data compiled by the Isenberg study indicate a definite relationship between failure of corrosion-weakened pipe and seismic shaking (081).

Evidence from the 1952 Kern County earthquake concerning leaks to steel pipelines ranging in size from 1-39 inches (25-990 mm) in Los Angeles also supports the corrosion-seismic shaking failure mode. Some 67 leaks resulted at rust holes on steel pipelines as a result of the earthquake (007).

Surface Fracturing and Ground Failure Effects on Buried Pipelines

Surface fracturing (tectonic movement associated with fault displacement) and ground failure (landslides, liquefaction, etc.) are the other major causes of failure of buried distribution and collection piping.

Buried pipelines are supported by the surrounding soil strata. Surface fracturing consists of relative movement of soil masses. If these soil masses

TABLE IV-7 - WATER MAIN DAMAGES:

SANTA ROSA EARTHQUAKES OF 1969 (015)

Location	Pipe material	Year installed	Damage Remarks
Sonoma Ave. (5 locations Sotoyome Ave. to Farmers Lane).	12" cast iron	1906	Pipe sheared 3" from joint end. No pipe displacement occurred.
Sonoma Ave. (2 locations).	16" steel	1939	Various small holes 1/2" diameter. No pipe displacement.
Talbot Ave. and Leonard (2 locations).	10" steel	1939	Small diameter holes. No pipe displacement.
Norte Way and Grahn Dr.	8" cast iron	1955	Four 3/4" bolts sheared on mechanical joint. Pipe displaced on alinement.
Wheeler St.	4" cast iron	1945	Crack in joint. No pipe displacement.
Salem Ave. (2 locations).	6" steel	1947	Various small breaks in side walls and existing leak bands.
La Crosse Ave.	6" asbestos cement pipe	1964	3/4" corporation stop threaded into pipe. Blown out of asbestos cement pipe. Minor damage to top of pipe. No pipe displacement.
Brookside Dr. (2 locations).	2" galvanized iron	1946	Side walls of pipe split.
Doyle Park Dr.	2" steel	1947	Pipe sheared next to existing repair clamp.
Polk St.	2" steel	1925	Small hole.
Bridle Trail	6" steel	1947	Various small holes.
Gilbert Dr.	2" steel	1949	Side wall of pipe split.
North St.	3" steel	1900	Hole in side wall.
Buena Vista Dr.	2" steel	1930	Small hole. Street damage occurred at leak location.

are supporting pipelines, the pipeline segments will also move relative to one another, inducing axial, bending and shear stresses on the pipe and possible failure. Pipe failure would be dependent on the pipe flexibility, surrounding soil parameters and the magnitude of relative movement.

Ground failure induced by seismic shaking may consist of liquefaction, landslides (caused by liquefaction) or soil consolidation. The soil failure may allow movement of large masses of soil taking any buried piping with it, causing pipe failure at soil mass interfaces. The soil immediately surrounding the pipe may liquify, removing the pipe support and causing a buoyant force to act on the pipe. Unsupported, the pipe may move in any direction, including floating upward.

Most of damage to the water distribution and sewage collection system of Niigata, Japan during the 1964 Niigata earthquake was a direct result of liquefaction, resulting in ground upheaval and uneven subsidence. The soil strata in Niigata consisted of sand and silt estuary deposits often extending to significant depths (as much as 15 meters near the Shinano River) (011). The groundwater level in the area was also very high at the time of the earthquake. As a result, the earthquake caused extensive liquefaction in the area, which generated large vertical (as high as 2 meters) and horizontal ground movements. Because of the large buoyant forces exerted on gravity flow concrete sewers by liquefaction, several pipelines were completely broken or cracked and joints were separated. Liquefied soil also exerted buoyant forces on manholes, causing them to float up above grade. Consequently, sewer connections to the manholes were also broken (011, 012).

During the 1968 Tokachi-Oki earthquake (013, 005), landslides of road embankments damaged pipelines laid parallel and near the road in the city of Misawa, Japan. Most of the damage occurred at the joint in the form of joint breakage and loosening as a result of the bending and shearing stresses exerted on the pipeline by the landslides. Approximately 2.1 pipeline failures occurred per kilometer as a result of the earthquake in the Misawa area.

Many of the damages to the water distribution and sewage collection systems of Anchorage, Alaska due to the 1964 earthquake were direct results of surface fracturing and massive ground failures. The local soil conditions consisted of outwashed sand, gravel, some glacial till and clay. Ground fractures and fissures were prominent in unconsolidated soil deposit areas. Areas of terrain in Anchorage were broken with horsts and grabens. Evidence of liquefaction was also observed through the presence of sand boils. In the Turnagain Heights area, a massive landslide resulted in the destruction of 75 homes and the distribution and collection systems serving that area. Both joints and pipe bodies of cast iron, asbestos-cement and concrete pipelines were broken due to the shear exerted on the pipelines by surface fracturing. Connections to manholes were broken and the manholes themselves damaged by the differential movement due to liquefaction (078).

Surface fracturing also caused extensive damage to the water distribution system of Managua, Nicaragua during the 1972 earthquake (034, 018, 037). Large joint displacement and pipeline breakage was caused by surface faulting. Table IV-8 provides details of pipeline breakage of the water distribution system of Managua following the earthquake.

TABLE IV-8

DAMAGES TO THE WATER SYSTEM OF MANAGUA, NICARAGUA FOLLOWING THE 1972 EARTHQUAKE (034)

EARTHQUAKE/ AFFECTED AREA	TYPE OF PIPE	PIPE DIAMETER mm* (in.)	DESCRIPTION OF DAMAGES														
Managua Dec. 23, 1972	See Description of Damages		PIPE BREAKAGE														
			GALVANIZED IRON			PVC			CAST-IRON			ASBESTOS-CEMENT			TOTAL		
Managua, Nicaragua			L** km (mi.)	BREAKS	No./km (#/mi.)	L km (mi.)	BREAKS	No./km (#/mi.)	L km (mi.)	BREAKS	No./km (#/mi.)	L km (mi.)	BREAKS	No./km (#/mi.)	L km (mi.)	BREAKS	No./km (#/mi.)
		25 (1)	2 (1)	6	3 (6)	3 (2)	--	--	--	--	--	--	--	--	2 (1)	6	3† (6)
		50 (2)	49 (30)	55	1.12 (1.83)	74 (46)	--	--	2 (1)	0	0 (0)	--	--	--	51 (31)	55	1.08† (1.77)
		75 (3)	--	--	--	--	--	--	7 (4)	17	2.43 (4.25)	7 (4)	2	0.28 (0.50)	14 (8)	19	1.36 (2.38)
		100 (4)	--	--	--	--	--	--	22 (14)	43	1.95 (3.07)	210 (130)	271	1.29 (2.08)	232 (144)	314	1.35 (2.18)
		150 (6)	--	--	--	--	--	--	15 (9)	12	0.80 (1.31)	89 (55)	103	1.16 (1.87)	104 (64)	115	1.10 (1.80)
		200 (18)	--	--	--	--	--	--	7 (4)	29	4.14 (7.25)	20 (12)	15	0.75 (1.25)	27 (16)	44	1.63 (2.75)
		250 (10)	--	--	--	--	--	--	1 (1)	1	1 (1)	1 (1)	0	0 (0)	2 (2)	1	0.50 (0.50)
		300 (12)	--	--	--	--	--	--	37 (23)	5	0.14 (0.22)	6 (4)	2	0.33 (0.50)	43 (27)	7	0.16 (0.26)
		350 (14)	--	--	--	--	--	--	1 (1)	0	0 (0)	--	--	--	1 (1)	0	0 (0)
		400 (16)	--	--	--	--	--	--	16 (10)	3	0.19 (0.30)	--	--	--	16 (10)	3	0.19 (0.30)
		450 (18)	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
		600 (24)	--	--	--	--	--	--	12 (7)	8	0.67 (1.14)	--	--	--	12 (7)	8	0.67 (1.14)
		900 (36)	--	--	--	--	--	--	3 (2)	0	0 (0)	--	--	--	3 (2)	0	0 (0)
		TOTAL	51 (31)	61	1.20 (1.97)	77 (48)	--	--	123 (76)	118	0.96 (1.55)	333 (206)	393	1.18 (1.91)	507 (313)	572	1.13† (1.83)

* Millimeters are rounded to nearest 5 mm.

** Kilometers and miles are rounded to nearest whole number.

† Does not include damages to PVC pipe due to insufficient analytical data.

The San Fernando earthquake of 1971 is another example where damages caused by surface fracturing were observed. Vitrified clay sewer pipe joints were broken either by excessive deflection or compression. Where compression was observed, the bells were broken away from the pipes. Extreme pipeline compression of joints resulted when some blocks of the damaged area were shortened by as much as five feet, due to tectonic movement. Conversely, elongation of blocks by as much as two feet caused joint separation, and thus the watertight integrity of the joint was lost. Surface fracturing, especially in the Sylmar region of the San Fernando Valley, caused uplift of the terrain and altered the grade of the sewers in the area, altering flow of sewage to some extent in virtually all sewers in the area. However, because of the generally gradual gradation of uplift, no reversal of flow occurred in any sewer (064).

Stream Crossings

Stream crossings, including inverted syphons, have been damaged by earthquakes. These structures, through necessary, are constructed in areas where ground failure of river embankments and river beds due to liquefaction is not unusual. During the Niigata earthquake in 1964, steel inverted syphons that crossed the Shinano River were completely destroyed due to embankment settlement and river bed upheaval. The area along the river experienced extensive liquefaction (011).

The 1975 Haichen earthquake and 1976 Tangshan earthquake in China severely damaged many pipelines which crossed streams. Ground failure of river embankments exerted bending stresses on the pipeline and caused rupture of the joint and pipeline breakage. In one instance, the pipeline was so extensively compressed by the failure of embankment, that the pipeline was pushed up above the surface of the water (071).

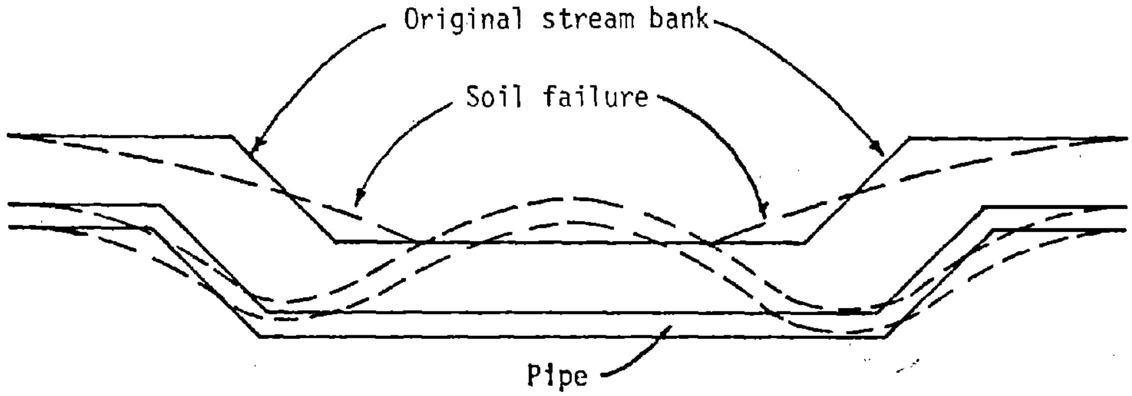


Figure IV-9. Typical stream crossing pipe failure.

Pressure Surges

Water hammer (pressure surges) in water distribution systems may be caused by the sudden closing of valves triggered by seismic motions or by earthquake accelerations of the contained water responding in hydraulic resonance. Young and Hunter (072) have shown, using a one-dimensonal analysis, that earthquake induced hydraulic pressure increases in water distribution systems may be as high as 435 psi, when subjected to a moderate earthquake. Water pressure surges in pipeline networks have been known to "blowout" water meter casings and vacuum breaker and air valve housings (033). Pressure surges have also caused blowouts in reaches of pipe weakened by corrosion.

C. TREATMENT SYSTEMS

The following discussion of potential damage to treatment systems includes damage to water and sewage treatment facilities with the exclusion of lagoons, which are beyond the scope of this report. This section includes a discussion of buried tankage, channels, buried piping and conduits, equipment, exposed piping and building structures. Treatment plant building structures are discussed only as they affect the operation of the treatment system. Emergency power sources and pump stations are included but are approached from a structural, rather than an operational viewpoint.

BURIED TANKAGE

Tanks are considered to be "buried" when the bottom of the tank lies below the ground surface. Buried concrete or steel tanks are found in most treatment systems. They typically represent the largest structure in the system. In sewage treatment plants, tanks may function as flow equalization basins, aerated grit chambers, primary, secondary and advanced waste treatment clarifiers, biological and chemical reactor basins and disinfection contact basins. In water treatment systems, aeration basins, mixing and flocculation tanks and clarifiers are typically constructed of concrete or steel. Filters in both water and sewage treatment plants may also be constructed of concrete or steel. Buried finished water reservoirs (clearwells), usually of concrete, are found in many water treatment systems.

Tank walls, internal components, foundations and appurtenances are all subject to earthquake induced failure through a variety of mechanisms. Some of these failure modes are discussed below.

Pressures on tank walls include outward impulsive (inertial) and convective (due to sloshing) pressures from liquids, as well as lateral pressures from surrounding soils. Tank walls are commonly designed as cantilever retaining walls to resist lateral earth pressures. A standard non-seismic tank design may include provisions for resisting static lateral earth pressure, groundwater pressure and flotation. An earthquake may cause the lateral earth pressure to increase through the inertia effect on the soil behind the retaining wall. Liquefaction can also occur as the result of an earthquake, causing the internal angle of friction in the soil behind the retaining wall to be effectively reduced to zero; the resulting lateral force exerted will be that of a liquid. Liquefaction potential may be high in uniformly graded, non-cohesive soils where groundwater is high.

The inertia of the mass of the tank structure or soils directly supported by the tank must be taken into account. If an acceleration of one-half of that of gravity is exerted in the tank overburden, the results could be catastrophic if adequate consideration and revisions were not made in design of the tank.

Impulsive and convective pressures of liquid contained in the tank exert lateral forces on interior tank components such as baffles, distribution and collection troughs, aerators, piping, etc. which may also be damaged. The inertia of the mass of the actual components may in some cases exert a substantial lateral force.

Because tanks are often massive structures, the integrity of the foundation is critical. While an earthquake would have little effect on the soil pressures from the foundation, the soil bearing capacity may change significantly. Vibration of soils with a low relative density such as fill or alluvial material may cause the soil to consolidate. Liquefaction of the underlying strata may cause the soil bearing capacity to be reduced substantially. Either of these may lead to uneven settling of tank structures, causing cracking and spalling which may be so severe that gravity flow through the plant would be prevented or sharply reduced. Liquefaction of soil surrounding an empty tank may even lead to the flotation of the tank.

When tanks settle, attached piping and feed and effluent channel connections are very vulnerable. If inlet or outlet devices are broken, the tank may be rendered inoperable, even though the tank itself is structurally sound.

The most extensive earthquake damage to a water treatment system documented in the literature was sustained by the Joseph Jensen Water Filtration plant of the Metropolitan Water District of Southern California. The Jensen treatment plant was under construction and only 85% complete at the time of the 1971 San Fernando earthquake. A major earthslide occurred at the plant site, covering an area 2500 feet by 800 feet. The area involved moved three to five feet laterally. A pressure ridge one to two feet high and about five feet wide developed at the base of the slide. Several sand boils from liquefaction appeared in the vicinity of the pressure ridge (033). The fill area experiencing sliding had a soil relative density of about 50% (082). It is estimated that this area experienced a horizontal acceleration of about 0.4 times gravity (033). Existing structures in the northeast section of the plant moved one-half foot to one foot, causing many expansion joints to open (033).

Mixing and settling basins founded on compacted fill in the northwest section suffered uneven settlement directly proportional to the depth of fill on which they were supported; the maximum settlement experienced was five inches. This led to the opening of expansion joints accompanied by concrete spalling. Unattached launders fell off columns, and sludge collector traveling bridge wheels jumped off tracks caused by shaking.

The most significant damage at the Jensen treatment plant was the failure of the finished water reservoir concrete structure. The reservoir is 520 feet by 500 feet, with a maximum water depth of 35 feet. The roof is supported by concrete columns 20 feet on center in both directions. The reservoir roof was to have been covered with seven feet of fill to prevent potential flotation of the empty tank. At the time of the earthquake, the groundwater table was at its maximum level and only two-thirds of the fill was in place. The failure of the structure is purported to have resulted from the inertia effect of the soil overburden. Shear pressures on the roof diaphragm of 450-500 psi caused failure of the diaphragm. The roof transferred the load to the reservoir walls, causing them to fail in bending. The floor and walls underwent differential settlement of three inches to six inches, although this is not believed to be a significant cause of structural failure (033).

Extensive damage to water and sewage treatment tanks occurred in El Centro, California, during the 1979 Imperial Valley earthquake (029). The most severely damaged facility was the water treatment plant's reactor-type flocculator-clarifier. The supporting members of the reactor unit were pulled from the tank wall anchors located along the bottomside of the peripheral wall. Several compression members within the reactor section and weir support members buckled.

The wastewater treatment facilities in El Centro consisted of primary and secondary biological treatment units. The primary clarifiers were older, above-ground units. The secondary facilities were newer, in-ground aeration tanks with platform mounted aerators with two secondary clarifiers. Damages were limited to the two secondary clarifiers and were primarily related to failure of the center wells and the associated impact on rake arms, skimmers and sludge withdrawal piping. Mixed liquor spills from sloshing were evident at both clarifiers in a northwesterly direction, in line with the epicenter location.

Extensive damage to clarifier No. 1 resulted from the failure of the center well support frame and the dropping of the center well over the sludge rake arms. As a result, the skimmers were pulled downward from the track and sludge withdrawal pipes were disconnected at the flexible hose connection elbow. Damages to the No. 2 clarifier were less severe and were limited to tilting of the center well structure. The steel support channel sections in this clarifier did not fail, and therefore, the center well remained in place. However, the angle sections used to support the center well bent as a result of the earthquake.

Both clarifiers were taken out of service because of the excessive drag of the rake arms and the resulting torque overloads on the rake arm drive motors. In general, the earthquake did not result in complete failure of wastewater treatment plant operations, although the treatment efficiency was greatly reduced due to the failure of the secondary facilities. The primary facilities remained operating after the earthquake with no problems reported.

Earthquake induced damages to water and wastewater treatment plant tanks also occurred in Peru (1974), Tokachi-Oki, Japan (1968), Niigata, Japan (1964) and San Francisco (1957). Failure of concrete tank walls in mixing tanks of the primary sedimentation tanks (see Figure IV-10, page IV-53), pipe breakage in the decanting tank and cracking of walls in the filter control building occurred in the Rimac River water treatment plant serving Lima, Peru (083). Two of four old concrete storage tanks cracked as they did in the 1970 earthquake. One new 26-foot deep reservoir cracked at the corners. The broken sedimentation tank wall is shown in Figure IV-10. The forces on the wall must have been from the water (no soil containment), and that failure occurred where the wall was laterally restrained. In Tokachi-Oki, brick masonry basins and filters cracked, a sludge digester settled unevenly and failed and a sewage treatment plant grit chamber cracked (013). Differential settlement of reclaimed land was also responsible for much of the damage in Niigata, including damage to a finished water reservoir founded partially on fill, and to a slow sand filter which cracked and inclined slightly (011). In some areas, liquefaction led to the flotation of small sewage treatment tanks (probably septic tanks), where attached piping broke off. Although the tank structures themselves suffered little damage, they were unable to operate out of position (084). Liquefaction also led to differential settlement of the pump suction well and combination aeration-upflow clarifier tanks in the Niigata sewage treatment plant (011). A sewage treatment plant grit chamber broke in two or three places (011). Three water filtration plants founded on firm ground suffered little damage (012).

In the 1975 San Francisco earthquake, a partially buried finished water concrete reservoir in Daly City developed a vertical crack in the middle of one of its side walls (008). It appears this crack developed due to increased lateral earth pressure on the side of the tank.

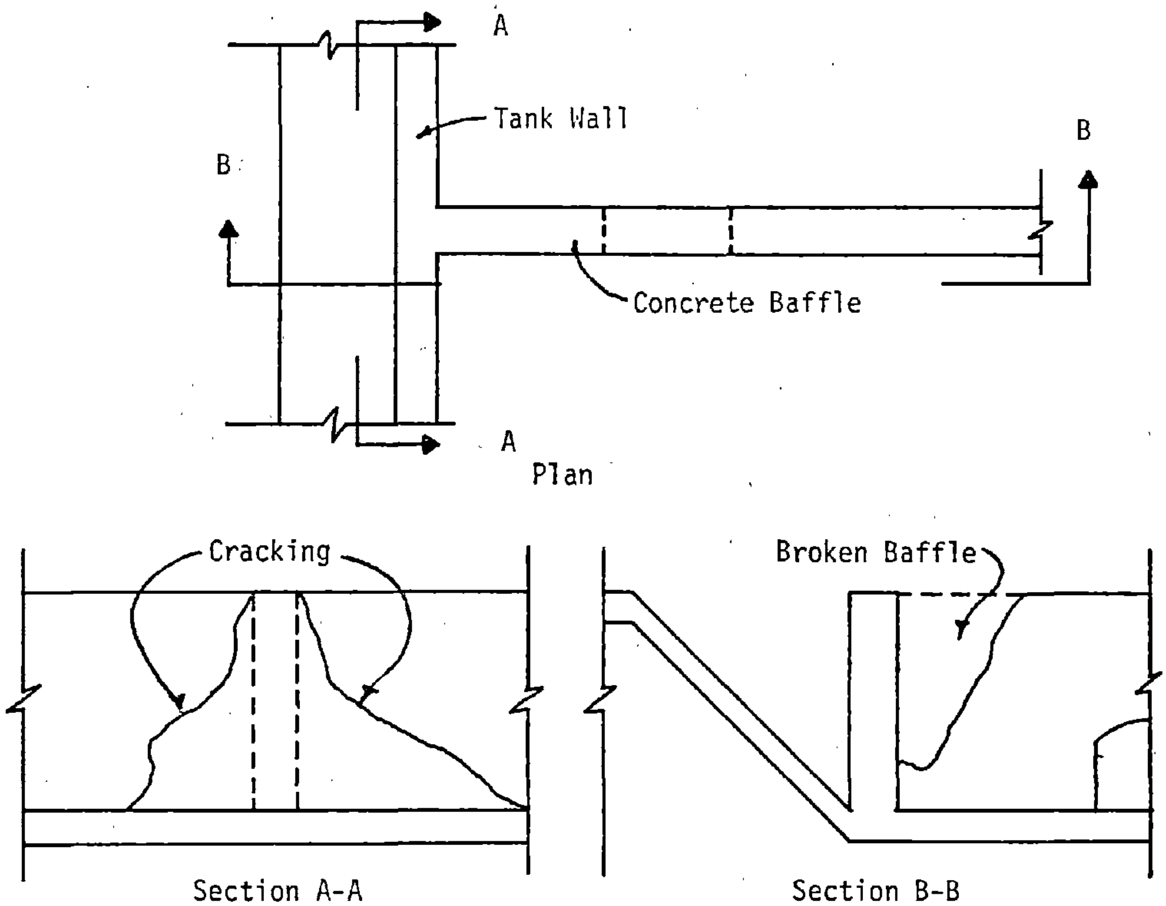


Figure IV-10. Rimac River sedimentation tank damage (083).

CHANNELS, BURIED PIPING AND CONDUITS

Channels, buried piping, and conduits suffer from earthquakes in much the same manner as buried tankage. Differential settlement from soil densification or liquefaction of the supporting strata can cause cracking and spalling of concrete. Differential lateral movement of tanks connected by channels or piping may cause joints to separate or push together, crushing the joint. Axial waves (primary waves) generated by the earthquake may induce axial strains on channels or piping (discussed in detail in Section IV B). Lateral earth pressure on the sides of open channels or box culvert walls may cause their failure in bending or shear. Connections to tanks may crack or spall due to differential movement or vibration.

The following damages were observed in the Joseph Jensen water filtration plant as a result of the 1971 San Fernando earthquake (033):

- 1/2-inch to 3-inch openings in the joints of effluent and overflow conduits immediately adjacent to the finished water reservoir
- failure in lateral shear of a 300-foot section of effluent conduit underlain with alluvium with 20 feet of overburden, causing a lateral deflection of 3 inches
- opening and spalling of expansion joints due to one-foot settlement of influent and connecting conduits to mixing basins, in the Jensen plant; voids were found under these conduit foundations

TREATMENT PLANT, PUMP STATION AND EMERGENCY POWER GENERATION EQUIPMENT

Earthquake motions may cause numerous modes of failure to treatment plant, pump station, and emergency power generation equipment. The greatest damage has historically occurred when earthquake induced forces have not been taken into account during design and installation. Equipment is secured from lateral movement by friction only, which may be reduced substantially during an earthquake due to vertical acceleration and horizontal forces on the equipment. When equipment moves or overturns, connections such as electrical conduit and piping can easily break. Horizontal circular tanks, although stable in one direction, can easily roll in the other if they are not properly anchored. Equipment moving off its foundation can itself be damaged or can cause adjacent equipment or structures to break when they interact.

Equipment anchorage systems often fail when anchor bolts are too small or when the concrete embedment cannot resist earthquake induced horizontal shear and overturning forces. If voids are left in the grouting under the equipment base, the grouting can be broken out during an earthquake if the equipment is not properly anchored.

Unanchored equipment damaged during past earthquakes include:

a. Lime storage tanks at the Sendai sewage treatment plant (Miyagiken-Oki, 1978) (085)

b. Eklutna power project transformers (Alaska, 1964) (036)

c. A hot water tank at Central Junior High School (Alaska, 1964) (086)

d. A 25-ton bottle washer at a soft drink plant, which caused connecting piping to break (Managua, 1978) (087)

e. Chlorine cylinders in Managua's water supply facility, resulting in breakage of connecting lines (034)

f. LADWP's (Los Angeles Department of Water & Power) chlorine scale at the Granada Chlorination Station, which was lifted from its pit (033)

g. Collector flights in the final settling tanks of the LA Hyperion treatment plant, which were shaken off their rails (033)

h. Thirty-one power transformers at the Sylmar Convertor Station, which caused leaks to develop (San Fernando, 1971) (033)

i. Unanchored equipment movement caused secondary damage such as breaking of piping and electrical connections in San Fernando (088)

Anchored equipment, on the other hand, survived past earthquakes quite well. In the Managua earthquake of 1972, in which a horizontal acceleration of 0.39 times gravity was experienced, a diesel generator, motor control center, pumps and miscellaneous heavy equipment anchored to a base slab at an oil refinery were undamaged (087). Anchored equipment in a Managua soft drink plant was damaged only as a result of debris falling from the collapsed roof (087). Major equipment with anchors designed to resist 0.1 gravity of horizontal acceleration did not suffer at the ENALUF Power Plant in Managua (1972) (035). Securing of chlorine tanks in the LADWP system prevented chlorine gas from leaking by preventing chlorine tank damage (San Fernando, 1971) (033). An anchored hot water tank at Elmendorf Air Force Base in Alaska (1964), similar in size to the unanchored tank at Central High School which toppled, did not move (086).

Vibration isolation systems including spring and rubber mounts have a significantly higher failure rate than rigidly anchored systems. Equipment such as blowers are commonly mounted on these systems to reduce operating noise levels in adjoining areas. For the system to effectively filter out high frequency vibrations, it must be flexible; hence, the horizontal restraint must be relatively weak. If the system is not designed with snubbers to limit lateral movement, it may easily fail under seismic motion. Vibration isolation system failure is often attributable to the fact that the system is anchored to a piece of equipment only, and not to the floor.

In the 1964 Alaska earthquake, motor/generator vibration isolation mounts permitted movement of the equipment since they were not bolted to the floor (086). A survey of Managua's industry after the earthquake showed that spring or rubber vibration isolation mountings failed in all cases except where pumps were mounted on inertia blocks keyed to the foundation, with springs underneath. Keying of the blocks to the foundation behaved as a snubber, limiting horizontal movement (035).

In the 1971 San Fernando earthquake, systems without vibration isolation systems generally suffered less damage than those with isolation systems. Most damage occurred when vibration isolation systems were not bolted to both the equipment and the floor. Some isolators were torn apart. An emergency generator supported on a multi-spring vibration system collapsed. The isolators were destroyed when cast iron spring guards failed, allowing the springs to pop out even though the system was "properly" mounted. It is interesting to note that molded neoprene isolators survived with practically no damage (088).

Equipment and small tanks mounted on legs are susceptible to failure during a seismic event. Earthquake induced forces are not typically taken into account in their design. Overturning and vertical acceleration forces can significantly increase the loading on equipment legs. Rocking of unanchored equipment can amplify the earthquake induced motions. Cast iron legs have little ductility and are easily broken under the impact of rocking.

The Managua industrial survey indicated that jack-type equipment legs moved since they lacked provisions for anchorage (087) and were unable to transfer shear to the equipment. In the 1964 Alaska earthquake at Fort Richardson, four cast iron legs supporting a sand filter, which were designed for static loading, failed (086). Numerous small tank leg failures occurred during the 1971 San Fernando earthquake (088).

Relative movement between flexible equipment and connecting systems can result in substantial damage. Out of phase vibration between two connected pieces of equipment can overstress the equipment and cause failure even if adequate anchoring has been provided. Banging between equipment abutting or close to a wall or another piece of equipment has been known to occur. Minor differential movement between a motor and pump, for example, can cause extensive damage if the system is operating during an earthquake event.

In the Port of Whittier, Alaska, a turbine generator mounted on a pedestal tied directly to rock displaced differentially with respect to the building surrounding it. Because differential movement was considered in the design, the only resulting damage was that of the electrical conduits between the building and the generator (040). At the Managua Thermal Electric Power Plant, a generator shaft became misaligned. Each of three turbines mounted on rigid concrete support structures moved in relation to the surrounding floor, causing superficial damage (089,035). A survey of industry after the 1964 Niigata earthquake indicated no damage to motor shafts when the motor was mounted on the same base as the connected equipment (084). Grain bins buckled at the National Grainery in Managua due to differential vibration responses of adjoining structures with different structural characteristics (035).

Flexible overhead power supplies in some facilities limited failure of electrical connections from movement of equipment during the Managua earthquake (087). A recommendation to allow adequate slack in electrical connections followed the 1923 earthquake in Kanto, Japan (090).

Failure of the equipment itself can be a major problem. There is little evidence of failure in heavy cast type equipment such as pumps and blowers, which have a low center of gravity. Taller pieces of equipment and their components have, however, been damaged during earthquakes; typical examples include tall reactor columns, cabinet-mounted equipment such as electronic instrumentation, and chemical feeders. Damages have included circuit board mounting failure and buckling of sheet metal cabinets and containers. Brittle structural components such as refractory material in incinerators and boilers and ceramic insulators have broken on many occasions. Structures supported over a relatively long span have failed as a result of differential settling of the foundation. Close tolerances must be maintained within active equipment (equipment designed to rotate or move) to prevent damage during an earthquake event.

Damage to storage tanks during past earthquakes includes failure of a fibre glass reinforced plastic tank storing potable water in Miyagiken-Oki, and the destruction of five fibre glass alum storage tanks at five different locations during the San Fernando earthquake (033). Chemical storage tanks cracked while settling four to six inches at the Jensen Water Treatment Plant (033). Differential settlement of a fuel storage tank located partially on fill and partially on piling led to its failure at the Managua Thermal Electric Power Plant (089, 035).

Breakage of stored material such as equipment replacement parts may be critical if they are required in the post-earthquake recovery period. Destruction of storage containers containing hazardous chemicals may endanger the life and safety of the facility personnel. Overturned battery storage racks, which damage or destroy the batteries used for emergency power, have significantly curtailed past earthquake recovery efforts.

Other examples of damage to equipment include damage to tall, lightweight structures such as washing and cooling towers in Niigata (084); breakage of refractory liners at the ENALUF Power Plant in Managua (035); and collapse of stacks composed of heavy refractory material in Alaska (properly guyed, lightweight double walled vents were not damaged) (086).

The failure of electrical systems in treatment or pumping facilities can lead to severe operating problems. Secondary insulators in the main service transformers serving Managua's water supply system failed (087). Numerous internal electrical components were broken in Managua's industrial facilities (035). In the power plant at Fort Richardson in Alaska (1964), many motors were burned out, damaged by falling debris. Most burnouts probably resulted from the starting of motors under low voltage conditions (086).

Equipment systems often rely on secondary systems such as lubrication pumps, batteries for startup and cooling or sealing water. While failure of one of these secondary systems may in itself be minor, the effect on the overall system could be very serious. During the 1972 Managua earthquake, diesel generators used for standby power at the Managua Thermal Electric Plant were inoperable because of damage to several support systems: the fuel tank overturned; the cooling water lines to 3 units broke at pipe joints; compressed air for the backup starting system had not been stored, nor was there a way to generate it; and one exhaust system was crushed (089,035). At the ENALUF Power Plant in Managua, the turbine support systems failed. Batteries used for supplying backup power to the oil lubrication pumps and valve controls fell off their racks. The turbine was damaged extensively because lubricating oil for its bearings was not delivered (035). An emergency generator at the Sendai sewage treatment plant moved six inches during the Miyagiken-Oki earthquake, breaking some electrical connections. Cooling water for the engine could not be supplied because its source, the public water supply system, had been rendered inoperable in that part of the City by the earthquake (085).

Secondary damage occurs when failure of one structure leads to damage of another. Even the most carefully seismically designed piece of equipment will be unable to survive an earthquake if, for example, a roof collapses on it; this occurred in a soft drink plant in Managua, where a falling roof damaged otherwise intact equipment (035). An overhead bridge crane in the Managua Thermal Electric Plant fell off its rails onto generators below (089,035). In the 1964 Alaska earthquake, counter weights came off of guide rails in elevator shafts, causing damage to structures while they were swinging (086).

The collapse of the east outlet structure in LADWP's Lower Van Norman Reservoir allowed sand gravel and rocks to enter the distribution system (San Fernando, 1971). All pumps receiving water from that reservoir were damaged by sand in the pump packing and seals. Bearings were burned out when lubrication oil was flushed out by water from leaking seals. The material plugged controls, control lines, surge suppressors, flow meters, pressure recorders, pump impellers, strainers and pressure regulators.

Table IV-9 summarizes the damages sustained by 11 sewage pumping stations in Sendai during the 1978 Miyagiken-Oki earthquake (022). Note that:

- All experienced a general power failure

TABLE IV-9. DAMAGE TO PUMPING STATIONS IN SENDAI (022)

Effects of the Earthquake
- Power Outages
- Suspension of Water Supply
- Faults in Emergency Power Units
- Faults in Electrical and/or Mechanical Systems
- Damage to Discharge Pipes
- Damage to Inlet Pipes
- Damage to Buildings and Other Equipment

Pumping Station	Drainage Area (ha)	Capacity (m^3/s)		Power Outages	Suspension of Water Supply	Faults in Emergency Power Units	Faults in Electrical and/or Mechanical Systems	Damage to Discharge Pipes	Damage to Inlet Pipes	Damage to Buildings and Other Equipment	Amount of Sewage Discharged into Waterways (m^3)	Normal Operation Resumed at
		Sewage	Storm Water									
A	55.84	0.134	0.860	X				?		X	Normal Operation Continued	
B	16.60	0.014	--	X							Negligible	0:30 on June 13
C	243.54	0.510	--	X	X →	X		X			2,000	1:30 on June 13
D	158.78	0.067	--	X					X		2,000	16:00 on June 14
E	920.00	0.419	7.687	X			X	X		X	450,000	16:25 on June 23
F	121.52	1.007	3.620	X		X	X	X		X	10,000	17:30 on June 14
G	139.30	0.135	--	X	X →	X		X	X	X	5,500	10:00 on June 14
H	168.00	0.149	--	X	X →	X					Negligible	10:00 on June 13
I	320.42	0.236	2.118	X					X	X	Normal Operation Continued	
J	53.96	0.033	--	X	X →	X					*	10:00 on June 13
K	108.00	0.151	--	X	X →	X	X			X	3,000	9:00 on June 13

* Discharged to Station K by gravity flow.

- Five were down because public water used to cool emergency power generators was not available
- One emergency power generator was damaged
- Three had problems with electrical or mechanical equipment
- Four experienced damage to discharge pipes
- Three experienced damage to intake pipes
- Six experienced damage to the pump station buildings or other equipment

Only two pumping stations were able to function normally. Malfunctions in the others resulted in the discharge of almost 500,000 m^3 (125 million gallons) of raw sewage into rivers flowing through Sendai. Pump stations were inoperative from 30 minutes to over 17 hours after the earthquake occurred.

TREATMENT PLANT PIPING

Treatment plant piping is considered to be exposed piping supported on pipe hangars or blocks. Pipe failure from earthquake induced motions can result from either differential movement between two systems or vibratory motions of the pipe itself.

Differential movement may occur in many situations. Sections of buildings may move relative to one another at expansion joints or failure planes. Pipe systems may move relative to the wall through which they pass. One piping system may move in relation to another where two large mass systems are connected by a relatively flexible link. Equipment may move differentially with respect to connected piping.

Earthquake vibrations include cyclic horizontal and vertical loads on piping systems. Failure may occur if pipe spans and pipe hangars are not designed to resist these additional loads. Piping systems react as continuous beams supported periodically. Under cyclic loading, the systems may react in various modes of vibration with the support points acting as modes. If allowed to vibrate substantially, stress building up at system discontinuities such as elbows, massive valves, attachments to equipment, wall penetrations and dissimilar points of restraint can result in failure at the weakest link, typically a joint or special fitting. Historically, many failures have occurred at inadequately designed flexible or expansion joints.

Failures in connections between pipes and equipment or among pipe sections have been observed during many earthquakes. Differential settling caused many utility connections to fail in Alaska (086), and broken building connections due to differential settlement were common in Niigata (091). In the 1971 San Fernando earthquake, many above-ground pipe failures were caused from differential displacement between equipment and piping (088). Many broken joints in concrete piping occurred in the Sendai sewage treatment plant as a result of the 1978 earthquake in Miyagiken-Oki (085). Some failures occurred in gasketed joints at the Managua Thermal Electric Power Plant, and some pipe breakage occurred at boilers, but the piping system generally performed well (089). Piping at higher building elevations in the power plant experienced greater movement and suffered greater damage (087).

Experience in Alaska (1964), Niigata (1969) and San Fernando (1971) has shown that while welded, soldered and brazed joints and mechanical couplings have survived earthquakes with relatively little damage, screwed joints have often failed at the joint threads (086,091,088). In Alaska, stress was developed in screwed fittings from the vibration of a long pipe section connected to a shorter leg (086).

Many earthquake induced failures in flexible joints have also been observed. In Alaska, flexible joints in cast iron pipe were pulled apart when the pipe was set in motion. Many bellows-type flexible pipe connections for thermal expansion failed due to lack of flexibility and the absence of pipe guides limiting lateral movement (086). Where flexible couplings were used between pumps and piping in a Los Angeles Department of Water and Power (LADWP) pumping station, no damage occurred (033).

The behavior of piping support systems in past earthquakes was varied. In Alaska, expansion loops in steam and hot water systems failed due to lack of bracing. As one pipe hangar failed, adjoining ones also failed due to the increased load (086). Piping and conduits suspended from the ceiling caused spalling of the plaster at anchor penetrations (040). Pipe support failure was reported at the Sendai sewage treatment plant as well. The piping support system at the Managua Thermal Electric Power Plant, on the other-hand, was designed for mechanical displacements with springs and snubbers, and the system functioned well (089).

Other earthquake induced damages in the literature include:

- Lifting of pumps by tension exerted from connected piping (Niigata, 1964) (091)
- Minor leakage in the pump discharge at Tujunga Galley Pumping Station, and in the pump section at the LADWP's Roxford Pumping Station (San Fernando, 1971) (033)
- Breakage of pipe between the wet and dry well walls at a pump station in Sendai (Miyagiken-Oki, 1978), which local authorities believed was caused by a water hammer, as a blank flange on the end of the pipe restricted the forward propagation of the shock wave (085)

- Shearing off of a valve behind the flange (Managua) (089)

It is significant to note that sprinkler systems installed in accordance with the National Fire Protection Code Standards performed well in Alaska (086).

Secondary impacts of pipeline failure can be extremely damaging. Flooding of facilities from broken water and sewer lines can severely damage electrical components. Shorted windings in motors require complete rebuilding of the motors. Instrumentation shorts can damage the complete system, requiring replacement of the electrical components as well as loss of pump control.

Damage of gas lines, which are found in treatment plants in both heating and anaerobic sludge digestion systems, can lead to damaging fires. Fires caused by gas leakage were minimized by the use of automatic shut-off valves in both Niigata (1964) (091) and Alaska (1964). In Alaska, the valves were designed to close when there was a pressure drop. Seismic activated valves (activated by earthquake accelerations) can also be used (086).

STRUCTURAL FAILURE-LOW PROFILE BUILDINGS

Building failure from earthquakes has received much attention in the earthquake engineering field. A detailed analysis of failure modes is beyond the scope of this report. Only the basic failure modes and their relation to treatment plant facilities will be presented here.

Building foundations may fail in a manner similar to tanks, suffering differential settlement from soil densification or liquefaction, which may shear connecting pipes and conduits. The building superstructure may fail from the earthquake vibration in many ways, depending on the type of design and construction. Rigid masonry buildings, for example, react quite differently from ductile steel frame buildings.

This discussion is primarily concerned with the destruction of equipment and facilities within the failing structure. Damage from falling objects such as light fixtures, ceilings, debris from roof failure, etc., may be extensive. Differential movement of the building foundation or superstructure may damage equipment supported by it. Systems supported in more than one location, such as piping systems, are vulnerable to this type of damage.

Immediately following an earthquake, access to all facilities for damage inspection is critical. Quick exit from the building may be required to insure the safety of personnel. Delivery of and access to stored materials may also be critical. Structural failure of the building or its components may block these access routes.

Examples of building failure during past earthquakes include:

- Broken walls in the filter control building of the Rimac water treatment plant in Lima (Peru, 1974) (092)
- Crumbling of block masonry chlorination building (Tokachi-Oki, 1968) (013)
- Broken windows and hairline cracks in masonry walls of the El Centro water treatment plant (Imperial Valley, 1979). This did not interfere with plant performance (029)
- Failure of structural members in the chemical building of the Joseph Jensen water treatment plant (San Fernando, 1971)
- Settling of the control building of 4 to 5 inches relative to undisturbed rock, causing a 2-inch differential from corner to corner, in the Joseph Jensen plant (San Fernando, 1971)

D. STORAGE TANKS

Damage to surface mounted and elevated water storage tanks is a common result of earthquakes. The loss of such facilities can seriously jeopardize the ability of a water supply system to provide sufficient water for fire protection, and to maintain a potable water supply with adequate pressure for the consumer. In addition, collapse of a tank could cause injuries and extensive property damage both from the falling structure and the rapid release of the tank contents.

The purpose of this section is to establish the possible modes of failure surface mounted and elevated tanks may experience when subjected to earthquake motions based on historical data. Damage to similar tanks found in the oil and wine industries will be included as well.

SURFACE MOUNTED TANKS

For the purpose of this discussion, surface mounted tanks, generally cylindrical in shape, are those whose bottoms are supported directly by the ground with little or no burial that could provide lateral support. The majority are constructed of steel plates, either welded or riveted (old design) together. There are, however, some reinforced concrete surface mounted tanks. Tank foundations may consist of simply treated gravel or sand layers, or may be a concrete ring wall supporting the tank walls.

The role of tank contents in causing failure seems to be clearly established. In the recent Diablo-Livermore earthquake (1980), 47 of 177 stainless steel tanks that were empty or only partially full at the Wente Brothers Winery suffered little or no damage. Of the remaining 130 tanks that were full, 96 suffered medium to severe damage (093). Damage reports from the Alaska (094) and San Fernando (033) earthquakes substantiate these findings.

Surface mounted tanks including their contents may be affected by earthquake motions in a number of different ways. The response of the water inside the tank is the primary driving force causing tanks to fail. The water inside the tank has been modeled based on the following response to earthquake horizontal motions (095): A portion of the water will move with the tank in short period motions; another portion of the water, primarily the top layer, will "slosh" back and forth across the tank in long period oscillations. Both of these responses will induce horizontal forces on the tank wall. In response to these forces, depending on their magnitude, the tank may slide or tip. One author noted that to his knowledge no tank larger than 40 feet in diameter with an H/D ratio less than one had ever slid due to ground shaking (043). The sloshing response may cause the tank to rock back and forth. The horizontal forces will exert a bending moment on the tank shell, exerting compressive stresses on the tank sidewall, at a maximum near the bottom of the tank. Water inside the tank is constantly exerting an outward static force on the tank wall in proportion to water depth. This loading may be amplified if the tank is subjected to vertical accelerations. With the compressive and outward forces acting on the tank wall simultaneously, it may bend outward, a phenomenon sometimes referred to as "elephant's foot" bulge. The stresses may be so extreme that the seam between plate sections may burst, allowing the discharge of water.

Another potential problem is tank foundation failure. One possible reason is the increased localized loading caused by the horizontal forces induced in the tank. The earthquake motions may cause the soil structure to break down and "liquify" or simply to compact, depending on the in-site soil conditions. This may allow the tank to tip or to settle unevenly, causing the tank shell or roof to buckle.

Reports from the San Fernando (033) and Imperial Valley (096) earthquakes indicate that tanks with rigid foundations, i.e., concrete ringwalls, are more likely to suffer from shell buckling than those with soft foundations, i.e., treated gravel or sand layers. This is probably due to increased localized stress concentrations, as the rigid concrete foundation will not deform.

Tanks are sometimes anchored to their foundations with bolts to resist rocking and sliding. Earthquakes have been known to stretch these bolts and even to rip the bolt connections out of the side of the tank. This again would allow the discharge of the tank's contents. In the 1978 Miyagiken-Oki earthquake, discharge of two oil tanks' contents was so rapid that a vacuum built up inside the tanks (lack of adequate air release) and caused the tanks to be crushed inward (085).

Tank roofs may buckle from the flexing of the tank walls. The horizontal and vertical accelerations to which the roof is subjected may cause an overload on the roof members or at the connection to the tank walls. Sloshing water may lift up portions of a tank roof, damaging either the roof or its attachment to the tank wall.

Pipes and other appurtenant items, such as stairways connected to the tank, may be broken loose due to tank movement. This movement could be caused by tank settlement, rocking or simply vibration that is out of phase with the adjacent ground to which the items may also be attached.

The height-to-diameter ratio seems to have an effect on the type and severity of damage a tank may incur. A conclusion drawn from the tank damage encountered in the San Fernando earthquake is that consideration should be given to keeping H/D ratio designs between 0.4 and 0.7. Observations made at the Southern Pacific Pipe Line tank farm following the Imperial Valley earthquake found that tank damage was heaviest to tanks with a H/D ratio greater than or equal to one.

Tanks with an H/D ratio between 0.8 and 1.5 at the Wente Brothers Winery suffered severe "elephant's foot" buckling around their full circumference. However, tanks with an H/D between 1.5 and 2.0 experienced a combination of "elephant's foot" shell buckling of less severity and/or diamond shape buckles, and no damage was found in those tanks with an H/D above 2.0. In tanks of this type light gauge (12 to 14 gauge) stainless steel, while damage was severe at H/D ratios between 0.8 and 1.5, damage decreased as the H/D ratio increased above 1.5 (093). Tanks with these larger H/D ratios are not commonly constructed for the water industry. They have, however, been used in the past and are commonly referred to as stand pipes. It should be noted that in accordance with the model presented in Chapter VII, a very small portion of the water responds in the sloshing mode in tanks with an H/D larger than 1.5.

Several surface mounted water storage tanks were damaged by the 1971 San Fernando earthquake. The Sesnon Tank of the LADWP is a welded steel structure, 92 feet in diameter and 42 feet in height with a maximum capacity of 2 million gallons. At the time of the earthquake, the tank held 1.9 million gallons of water. The tank was constructed on fill (40-53 feet deep) and designed under the L.A. Building Code to withstand a 0.2g horizontal earthquake loading. However, the fill did not meet the minimum 90 percent soil density requirement stipulated by the LADWP. Consequently, failure of the foundation and differential settlement caused by the earthquake resulted in a horizontal buckle in the shell plate, 24 feet above the bottom and approximately 150 degrees in circumference on the south side of the tank. In addition, a portion of the redwood sheathed roof was lifted, probably due to contact from the sloshing of the tank contents. The steel sketch plate (base plate) and foundation separated by as much as two inches in several places. The tank settled as much as nine inches due to the earthquake's consolidation of the fill on which it was founded (033).

The Granada High Tank of the LADWP system, a 55-foot diameter, 45-foot high riveted steel tank covered with wood supported by light-gauge steel angle trusses, was also damaged by the 1971 San Fernando earthquake. At the time of the earthquake, the tank held about 3/4 of its 586,000 gallon maximum capacity. Damage consisted of the collapse of the roof structure and separation of the tank at the base from the asphalt pavement surrounding the tank. The area experienced severe ground shaking as evidenced by cracked pavement and small landslides. The roof structure collapsed when the support members failed due to increased loading from earthquake accelerations (033).

The steel washwater tank located at the Joseph Jensen Filtration Plant measures 100 feet in diameter and 36-1/2 feet in height. At the time of the earthquake the tank was half full. The tank's foundation consisted of a concrete ring wall 14 inches thick and 3 feet deep, and the tank was located directly on undisturbed dense soils. The tank was anchored to the ring wall by 12 one-inch diameter anchor bolts, equally spaced about the perimeter of the tank. Sloshing of the tank contents set the tank into a rocking motion. The anchor bolts then either failed in tension or pulled out. The resulting impact of the tank base with the ring wall from the rocking motion caused buckling of the upper shell wall. The amount of vertical movement was indicated by the length of anchor bolt pulled from the foundation, as much as 13 inches on the south side of the tank. Other damage included stairway treads being broken away from the side of the tank (033).

Five other surface mounted storage tanks damaged by the 1971 San Fernando earthquake were located in Kagel Canyon (L.A. County Waterworks). The size of the tanks ranged from 15 to 27.5 feet in diameter and 18 to 24 feet in height. All were of welded steel construction. Horizontal and vertical movements generated by the earthquake caused slight displacement from the foundation, buckling of shells near the base and breakage of valves and fittings of attached piping of all the tanks. The tank contents of all five tanks were lost (033).

Considerable damage to surface mounted storage tanks occurred over a wide area of Alaska during the 1964 earthquake. A significant portion of the damage was caused by tsunamis and ground failure. However, this section will only discuss those damages directly attributable to ground shaking, which generated structural failure.

Table VI-10 lists a number of tanks, their characteristics and damages caused by the earthquake. These tanks stored both water and various fuels. However, the basic design of all the tanks and fluid properties of the tank contents were similar from a damage analysis viewpoint.

Design of the tanks did not take into consideration any seismic force loadings. Their basic configuration consisted of a cylindrical steel wall, welded to a thin flat steel bottom plate which rested on the ground, and a roof plate.

Analyses of the characteristics of damage reported for the tanks identified in Table VI-10 revealed the following types of failure (094):

- Total collapse of the tank - A water tank which was full at the time of the earthquake buckled 6-24 inches from the bottom plate. Consequently, the bottom of the tank ripped loose from the tank wall on

TABLE IV-10. TANK PROPERTIES AND DAMAGE CAUSED BY GROUND SHAKING FOLLOWING THE 1964 ALASKA EARTHQUAKE (094)

Tank	Diameter, (D) ft	Height (H) ft	Capacity, bbls	Condition at time of Earthquake	Damage Observed
A	30	48	-	Full of water	Collasped
B	100	32	44,700	Full of oil	Damage to roof, top wall, and roof columns
C	45	32	9,000	Full of turbine fuel	Damage to roof, top wall, and roof rafters and the bottom wall buckled
D	120	32	64,500	Full of oil	Damage to roof, top wall, and roof columns
E	120	32	64,500	Almost empty	No damage
F	120	32	64,500	Almost empty	No damage
G	110	32	54,000	Almost empty	No damage
H	90	32	36,100	2/3 full	No damage, except to the swing joint in the floating section
I	55	23	10,171	Full of fuel oil	Damage to roof rafters and top wall
J	30	40	5,000	Full	Extensive buckling of the bottom wall
K	30	40	5,000	Full	
L	30	40	5,000	Full	
M	28	40	4,388	Full	Collapsed
N	42	40	10,123	-	Buckled bottom wall
O	20	40	2,233	-	Bottom wall buckled and broke the wall-to-bottom-plate weld
P	144	56	-	-	Floating roof buckled; indica-cations of large waves
Q	112	56	-	-	Floating roof pon-toon damaged
R	49	48	-	-	Bottom wall buckled; indica-tions of 10-12 in. uplift of the tank
S	90	48	-	Over 3/4 full	Roof-top wall connection and roof structural steel damaged
T	160	56	200,000	-	Support columns twisted and rafters damaged
U	160	56	200,000	-	No damage

the side opposite of the buckle and the tank overturned. The cone roof was ripped off and propelled 75 yards in the direction of the collapse.

- Roof buckling - A number of column supported, steel cone-roofs buckled. This was thought to be caused by the combination of weight of heavy snowfall, water ponding and earthquake aftershocks.
- Failure at roof to shell connection - The roof to shell connection for most of the tanks was designed as a weak connection, to allow failure of the connection in the case of over filling.
- Shell buckling - Circumferential shell buckling occurred on many tanks as a result of rocking of the tank during the earthquake.

Storage tanks were also damaged by the 1972 Managua earthquake. Three thin-skinned steel water tanks sutained "elephant's foot" buckling. Inlet and outlet piping connections were also broken. Tanks that sustained such damage were demolished and reconstructed with welded steel according to AWWA standards (034).

The surge tank, containing about 2/3 of its 114,000 gallon capacity, located next to Asososca Booster Station in Managua, experienced failure of the anchor bolts. The anchor bolts were pulled from the foundation, due to the rocking motion of the tank (034, 035).

A number of water and fuel surface mounted storage tanks were damaged as a result of the 1964 Niigata earthquake. The major modes of failure are itemized below (097, 098, 084).

- Sloshing liquid damaged several tank roofs.
- Foundation failure-consolidation and liquefaction of soils allowed differential settlement and inclination of the tank. As a result,

inlet and outlet piping connections were broken and shell walls were buckled. Tanks supported on soils stabilized with vibro-flotation responded well. Concrete ringwall foundations limited differential settlement.

In the recent Imperial Valley earthquake, the City of Calexico's wastewater plant's surface-supported concrete sludge digester tank roof cracked and well/roof seal broke, allowing gas to escape (099). In the same earthquake, the Southern Pacific Pipe Lines tank farm north of El Centro suffered moderate damage. Tanks rocked on their foundations, breaking connecting lines and rupturing the sidewall/bottom seam. "Elephant's foot" buckling also took place (099).

ELEVATED STORAGE TANKS

Elevated storage tanks are generally either supported by a braced frame or a pedestal. The frames or pedestals are commonly constructed of steel. However, there are some concrete elevated pedestal tanks in use.

Elevated tanks may fail because of foundation failure or rupture of the tank itself. The primary failure mode encountered is the failure of the tank support structure. The tank structure will respond to horizontal earthquake motions essentially as a single degree of freedom system, i.e., a mass oscillating on a spring. While a portion of the water inside the tank may have an independent response, its effect is normally considered to be negligible. The system has a moderately long period. The earthquake horizontal accelerations will induce stress on the various members of the supporting structure (braced frame). The structure may be simultaneously subjected to vertical earthquake accelerations, responding as a rigid system. The stresses from both the horizontal and vertical accelerations would then be combined. If the stress induced in a member is greater than its yield stress, it will yield. If the member yields enough it will fail. Once a member has failed, it will transfer the loading it was resisting to other members with possible ultimate tank failure occurring by the "domino effect."

The tower structure, having a long period response, may have a large horizontal displacement, sometimes referred to as drift. This may substantially realign the loadings on the support structure which may not have been considered in the design. The effect of eccentric vertical loading on the support structure from the weight of the supported object at an extreme horizontal displacement is sometimes referred to as the P-delta effect. These relocated loadings may cause the supporting structure to fail.

Some probable causes of tank support structure failure include:

- Ripping of clevis or gusket steel bracing connections
- Shearing of bolts or pins at connections
- Spreading of clevises allowing pins to fall out
- Failure of tie rods at threads or other locations
- Bending of horizontal compression bracing

Other damage may include:

- Spalling and cracking of concrete foundation
- Stretching of anchor bolts (While anchor bolt failure has been noted, it was believed to have been a result of level action of falling columns and not directly from earthquake loadings (007).)

Support structure column buckling has historically not initiated failure. Following the 1952 Kern County earthquake where a number of elevated tanks suffered damage, direct column failure was not noted (007).

In the 1976 Friuli, Italy earthquake, an elevated tank rotated about ten degrees inside a saddle supported by a braced frame structure. This is believed to have been caused by water swirling inside the tank responding to random multi-directional earthquake motions. It was reported the support structure sheared eight to ten bolts at clevis connections while responding to the earthquake (021). The Friuli earthquake also caused a 44-foot high concrete pedestal-type tank to collapse, breaking off at the pedestal (021).

The recent Imperial Valley earthquake subjected eight to ten elevated tanks to ground movement. It was reported that of these, two were damaged and one collapsed. A gusset plate pulled out of a tubular column on one damaged tank with buckling of one horizontal strut. At El Centro, another tank's diagonal tie rods in the upper level of bracing stretched, horizontal compression members buckled and anchor bolts stretched. One 100,000-gallon elevated braced frame tank collapsed nearly within the bounds of its base. Failure is reported to have been initiated by cross bracing failure (096).

During the 1952 Kern County (Bakersfield, California) earthquake, 16 of 25 tanks in the area sustained some form of damage. Of 12 tanks designed to resist wind, two collapsed and seven suffered rod distortion or failure. The remaining tanks were designed to resist a horizontal earthquake acceleration ranging from 0.08 to 0.20 times gravity. Of these tanks, only one failed, with the others sustaining little or no damage. It was reported that the seismic resistant designed tank collapsed because of cotter key failure, i.e., either shearing or falling out (007).

CHAPTER V

SYSTEM SITING, PLANNING AND CONCEPTUAL DESIGN

Earthquake induced interruptions in water and wastewater services can be reduced with proper site planning and conceptual design. At a basic level, this requires a detailed knowledge of the geotechnical characteristics of potential construction sites such as location of fault zones and soil strata. Seismic design should also consider redundancy, operational flexibility, and ease of repair. Many of these considerations are similar to those for determining fire flow reliability for fire insurance ratings. Experience has shown that when such factors are considered during the planning stage, lengthy earthquake induced service interruptions have often been forestalled.

This chapter will present site planning and design strategies that can be used to decrease the seismic vulnerability of water and wastewater systems. Section A, "Seismic Resistant Siting Considerations", is specific to earthquake protection. Section B discusses design strategies applicable to system reliability and disaster planning in general, with a few specific references to earthquakes. System reliability using mathematical risk analysis techniques is discussed in Section C.

A. SEISMIC RESISTANT SITING CONSIDERATIONS

Facilities can be damaged by direct and indirect earthquake effects. Direct effects include fault displacement, uplift and subsidence, and seismic shaking; indirect effects include landslides, ground fractures, lateral soil displacement and differential compaction of sediments. Lateral displacement and differential settlement may be due to liquefaction (073).

Proper siting of water and wastewater facilities requires an understanding of the nature of these effects and their potential location. This section will discuss the types of geotechnical environments conducive to these effects, how to obtain site specific information, and how to apply this information to facility siting.

SURFACE FAULTING

A fault has been defined as "a fracture or fracture zone along which there has been displacement of the two sides relative to one another parallel to the fracture" (100). It can be classified as active, potentially active, or inactive. Faults which have shown historical activity are known as active, while those without recognized activity are termed inactive. Potentially active faults show strong indications of geologically recent activity, although available data do not indicate that historical ground ruptures have occurred (101).

The surface along a fault zone may or may not rupture during an earthquake. For a given geographic area, given an earthquake magnitude and location, it may be possible to estimate the likelihood of surface ruptures and the type of movement based on past earthquakes in the study area (047). The state of the art does not, however, enable an accurate prediction of the occurrence or extent of surface rupture (102).

Fault displacement can lead to changes in land level, known as tectonic uplift or subsidence. These movements can be restricted to local areas adjacent to the fault, or they can extend for several miles from the fault. The 1964 Alaska earthquake produced uplifts of as much as 38 feet and downwarps of more than 7 feet over an area of 110,000 square miles (103). This type of effect is significant with respect to aqueducts and other pipelines which operate through gravity flow.

GROUND SHAKING

Most earthquake induced damages have typically resulted from the direct and indirect effects of ground shaking. In California, for example, it is estimated that over 95 percent of all earthquake induced damages have resulted from shaking which damaged structures directly, led to soil failure beneath the foundations of structures, and/or caused the soil beneath the foundation to densify and settle, resulting in structural failure (101).

The direct effects of shaking are discussed elsewhere in this report. With respect to siting, however, it should be mentioned here that the factors influencing ground shaking are exceedingly complex. Shaking does not necessarily dissipate with distance from the epicenter; shorter period vibrations tend to be damped over distance, while longer period vibrations may travel farther. In Alaska, for example, the 1964 earthquake, with its epicenter in Prince William Sound, caused "extensive damage to tall buildings when low-frequency waves reached Anchorage, 80 miles away" (102). The depth of the focus also has an effect on the intensity of ground shaking at varying distances. When the focus is relatively shallow, typical of California earthquakes, the shaking intensity decreases relatively rapidly with increasing distance from the focus. Shaking intensity is less a function of distance with deep foci, typical of central and eastern United States earthquakes; effects are felt for much greater distances. The 1811 and 1812 New Madrid, Missouri earthquakes (Modified Mercali intensity of XII) were felt as far away as Washington, D.C. and Boston (104). Furthermore, shaking can be amplified or altered by site-specific conditions such as ground firmness and soil depth (102). Soft soils such as clay and loose sand may damp high frequency vibrations but may amplify low frequency vibrations at particular frequencies.

INDIRECT EARTHQUAKE EFFECTS

Indirect effects of earthquakes relate to the behavior of soils when subjected to seismic shaking and typically include soil compaction, liquefaction, lateral displacement, and landslides. These effects, collectively termed "ground failure", have been responsible for major catastrophes during past earthquakes.

Compaction of Non-Cohesive Soils

Ground vibrations caused by earthquakes often lead to compaction or densification of non-cohesive soils such as loose sand or gravel deposits or fill areas. This, in turn, leads to settlement of the ground surface. Ground settlement resulting from soil compaction often results in differential settlement of engineering structures, often causing the structure to fail. Differential settlement can occur when a supporting soil layer varies in thickness (leading to varying absolute amounts of compaction), or when a structure is founded on dissimilar soil strata. Uneven settlement of fill areas at the Joseph Jensen Water Filtration Plant caused extensive damage during the 1971 earthquake in the San Fernando Valley.

An example plot showing the expected settlement (percent of the alluvium layer) as a function of both earthquake magnitude and the distance from the causative fault for a typical condition is presented in Figure V-1. This information was developed for the Joseph Jensen Filtration plant after the San Fernando earthquake and is presented for illustration purposes only (105).

Soil Liquefaction

Liquefaction may occur when non-cohesive water-saturated soils are vibrated by cyclic shear stresses due to the upward propagation of shear waves through the soil deposits. This vibration is accompanied by a tendency for the soil to compact, resulting in an increase in pore water pressure and a reduction in stress on the soil grains. In the absence of shear stress between the soil particles, the shear strength of the soil is lost, and the soil transforms from a solid to a liquefied state. When the cyclic stress is discontinued, the residual pore water pressure on the soil will be equal to the overburden pressure, leading to the upward flow of water through the soil (106).

Liquefaction commonly leads to three basic types of ground failure: lateral spread, flow failure, and loss of bearing capacity from the soil's shear strength. Lateral spread involves the lateral movement of a surface layer due to liquefaction and loss of strength in an underlying layer. It commonly develops when liquefaction occurs on a 0.5 to 5 percent slope,

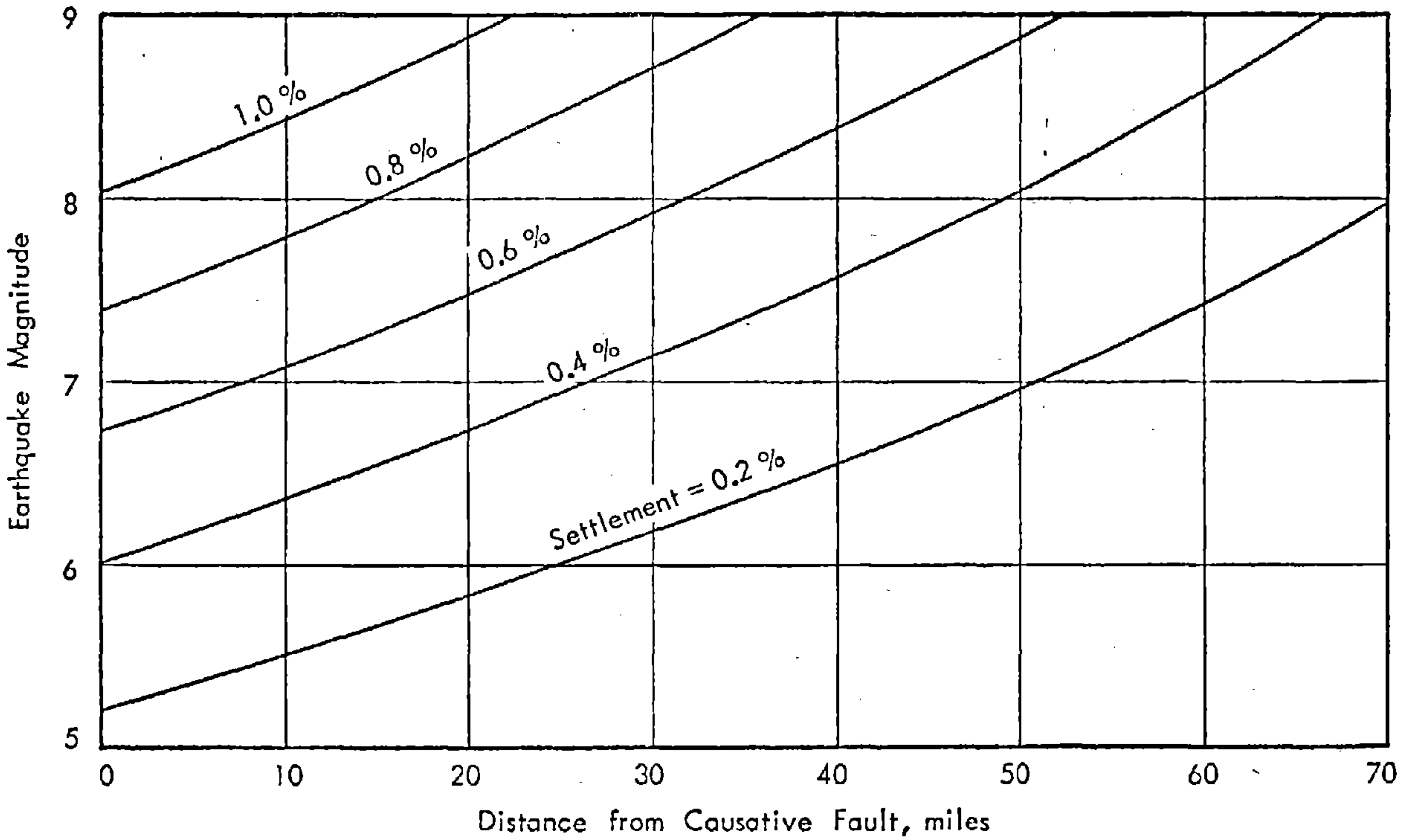


Figure V-1. Settlement in the unsaturated compacted fill with a dry density of 122 PCF when subjected to various earthquakes (The Joseph Jensen Filtration Plant) (105).

leading to lateral displacements of up to several feet. Rigid structures such as pipelines and buildings may be compressed or pulled apart on lateral spread areas, depending on their orientation. It is reported that "every major pipeline break in the City of San Francisco during the 1906 earthquake occurred in areas of lateral spreading" (107, 047). Much of the damage at the Joseph Jensen Water Filtration Plant was a result of lateral spread caused by the 1971 San Fernando earthquake (107, 033).

Flow failures can develop when liquefaction occurs in or under a soil mass sloping typically greater than five percent. The entire mass can flow laterally to the unsupported side at speeds of up to several tens of miles per hour for distances of tens of feet (108). Many catastrophes have been caused by massive flow failures during past earthquakes. In the 1920 Kansu, China earthquake, landslides (flow slides) were responsible for the majority of the 200,000 deaths and for the destruction of hundreds of towns and cities (108, 109). In the 1964 Alaska earthquake, 98,000 million $yds.^3$ of sediment moved in flow failure, destroying all the dock facilities at Valdez (107, 108, 110). Flow failures occurring in submerged conditions can lead to the formation of tsunamis (107).

Liquefaction also can cause loss of bearing capacity from the soils shear strength. When this occurs in a soil supporting a structure, large displacements can occur within the soil, allowing the structure to settle, tip, overturn or buoyantly rise (float). This type of ground failure was especially evident in Niigata, Japan, during the 1964 earthquake. Some Niigata soil conditions, liquefiable material beginning near the ground surface and extending to a depth greater than one-half the foundation width, allowed building overturning. Thinner or deeper liquefiable layers may allow settlement (107).

Strength Loss in Sensitive Clays

Although most clays lose strength when disturbed, those that suffer large strength losses are classified as sensitive. When clay is under pressure for extended periods, the particles become bonded by highly viscous water, which develops a high shear strength. When the strong bonds are broken by deformations caused by seismic shaking, less viscous water is drawn between the particles, reducing the shear strength of the soil. The particles can then be easily displaced (111). The results of sensitive clay soil failure could be comparable to liquefaction of non-cohesive soils.

The ratio of the strength of an intact soil specimen to the strength of the same specimen after it has been disturbed, determined at equal water content, is known as the sensitivity of the clay. Insensitive clays are characterized by a sensitivity of less than 4, sensitive clays by a sensitivity greater than 8. Clays with a sensitivity exceeding 10 may be prone to failure during strong seismic shaking (107). Sensitive clay failures with sensitivities between 10 and 40 were involved in five landslides disrupting Anchorage in the 1964 Alaska earthquake (107). St. Lawrence River clay, which has a sensitivity of about 20, suffered a breakdown in soil structure due to vibrations from equipment used during the construction of the St. Lawrence Seaway (111); earthquake vibrations in this highly seismic area could be more intense than vibrations from construction equipment.

Landslides

Liquefaction of non-cohesive soil is commonly associated with landslides (109). Soil failure on steep slopes may also play an important role (107). Landslide causes can be categorized as follows (109, 107):

1. Flow slides or flow failure

2. Liquefaction of thin, non-cohesive supporting strata
3. Liquefaction of sand lenses in sensitive clay layer supporting strata
4. Collapse of fills from liquefaction of foundation material
5. Rock or earth slides from fracturing brittle rock or loosened debris on steep slopes (greater than 50%) stressed by gravity (Peru 1970, killed 18,000 people, involved 65 million $yds.^3$ rock, travelling as fast as 200 mph) (107).

While liquefaction is usually involved in landslides on moderate or flat slopes, displacement of up to several feet may occur in marginally stable slopes, i.e., fills on sloping ground (107).

LIQUEFACTION POTENTIAL

Liquefaction potential is related to soil type, soil relative density or void ratio, overburden pressure, and intensity and duration of ground shaking (112). It generally occurs when the water table is within 30 feet of the ground surface and in recently deposited sediments such as river channels, flood plains, poorly compacted fills, and Holocene deltaic deposits (101).

The soil type in cohesionless soils can be characterized by grain size distribution. For uniformly graded soils, fine sands generally liquefy more easily than coarse sands, gravelly soils, silts, or clays (101). Laboratory testing indicates that soils with a D50 (mean grain size) between 0.02 mm and 0.4 mm are more vulnerable to liquefaction than finer or coarser materials (109). Another study indicates that soils with a D50 of 0.08 mm are the most susceptible to liquefaction (112). Opinions are conflicting regarding the relationship between liquefaction potential and the uniformity of soil grading. Evidence has been presented showing that uniformly graded soils (small variation in particle sizes) are more susceptible than well-graded soils (wide range of particle sizes) (112), while the graphs shown in Figure V-2 , used for designing port structures and bridges in Japan, indicate that well-graded soils are subject to liquefaction over a wider range of particle sizes than soils with uniform grading (113).

The relative density of a soil, D_r, is defined as $\frac{e_l - e}{e_l - e_d}$, where e is the void ratio of the sample, e_l is the void ratio in the loosest state, and e_d is the void ratio in the densest state (111). In general, the liquefaction

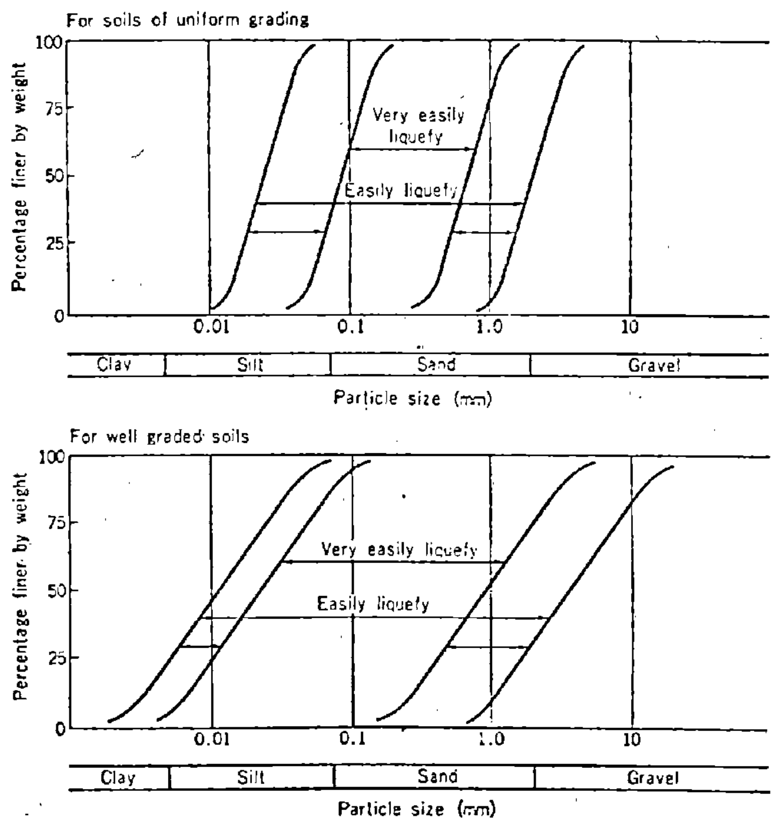


Figure V-2. Ranges of grain size accumulation curves of sands which have possibility of liquefaction (113).

potential of a soil decreases as its relative density increases (112). Relative density is sometimes approximated by measuring the soil's penetration resistance. The Standard Penetration Resistance Test, used in situ, measures the number of blows (N) required for a 140# hammer dropped 30" on a drill rod to drive a standard 2-inch O.D. split barrel samples 12 inches into the bottom of a soil bore hole (114). The relationship between N, D_r, and the effective overburden pressure is shown in Figure V-3. (Effective overburden pressure is based on the combined unit weights and depths of the overlying soil layers after accounting for the buoyant effect of water in the saturated layers. This is discussed in greater detail later). Depending on which parameter is being used to evaluate liquefaction potential (discussed later), this chart can be used to estimate D_r from N, or vice versa.

The liquefaction potential of a soil has been shown to decrease as overburden pressure increases. In the 1964 Niigata earthquake, soil underlying a 9-foot fill remained stable, while similar soils surrounding the fill suffered extensive liquefaction (112).

For a given type of soil under a given confining pressure, the liquefaction potential will be a function of the magnitude of stress induced in the soil and the number of stress cycles. The magnitude of stress depends on the intensity of ground shaking, and the number of significant stress cycles depends on the duration of shaking (112). For an accurate determination of liquefaction potential, both of these factors must be considered.

Evaluating the liquefaction potential of a deposit of saturated sand requires two basic steps: computing the cyclic stress ratio $\frac{\tau_h}{\sigma_o'}$ (defined later) that will develop in the soil deposit during an earthquake of a given magnitude, and then determining whether this cyclic stress ratio will cause lique-

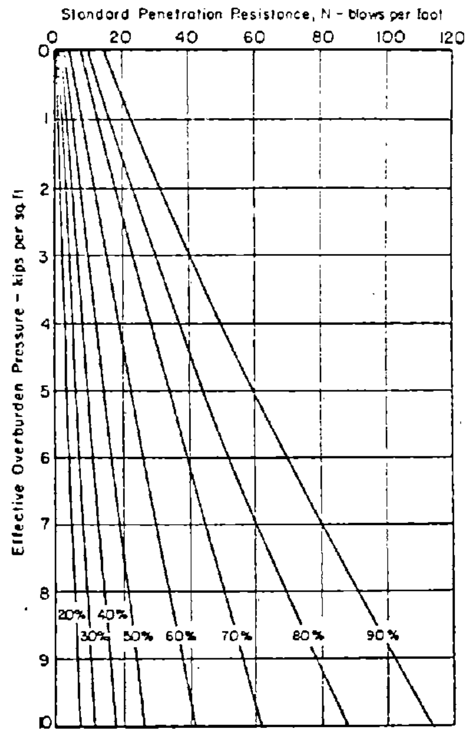


Figure V-3. Relationship between standard penetration resistance, relative density and effective overburden pressure (after Gibbs and Holtz) (112).

faction to occur in the soil deposit. The cyclic stress ratio that will develop in the field due to earthquake shaking can be computed mathematically or by ground response analysis. The cyclic stress ratio that will cause liquefaction can be determined by using an empirical field correlation approach, or be performing a simple shear test or cyclic loading triaxial test in the laboratory. A good discussion of liquefaction potential from which the majority of information in this section was obtained is included in "Evaluation of Soil Liquefaction Potential During Earthquakes" by Seed et al., 1974 (106).

The method presented below consists of using an equation to compute cyclic stress and then determining liquefaction potential through the empirical field correlation approach. As will be discussed later, this method is useful only as a rough indication of liquefaction potential at a particular site. If it reveals that liquefaction potential is borderline or high, more sophisticated methods should be applied. These will be described only briefly, but sources of detailed information will be provided.

The cyclic stress ratio $\frac{(\tau_h)_{av}}{\sigma_o'}$ that will develop in a soil deposit due to earthquake shaking can be computed from the following equation (106):

$$\frac{(\tau_h)_{av}}{\sigma_o'} \approx 0.65 \ \frac{A_{max}}{g} \cdot \frac{\sigma_o}{\sigma_o'} \cdot r_d$$

where:

$(\tau_h)_{av}$ = average cyclic shear stress developed on horizontal surfaces of the sand as a result of an earthquake

A_{max} = maximum acceleration at the ground surface resulting from an earthquake of a given magnitude (can be taken as the effective peak acceleration as shown in Figures VII-1 and VII-2)

g = gravity acceleration

σ_o = total overburden pressure on the sand layer under consideration

σ_o' = initial effective overburden pressure on sand layer under consideration

r_d = stress reduction factor

The total overburden pressure σ_o is calculated by multiplying the unit weight of each overlying soil layer by its depth and summing the respective products.

The initial effective overburden pressure σ_o' accounts for the buoyant effect of water on the soil layers below the groundwater table. It can be calculated with a knowledge of the specific gravities and void ratios of the submerged soil layers.

The stress reduction factor r_d accounts for the fact that the cyclic shear stress will decrease with depth because the soil column behaves as a deformable body. The r_d value is 1 at the surface; values at different depths are shown in Figure V-4 . Using the average values shown by the dashed line would result in about 5 percent error for depths up to 40 feet (112).

Thus, with a knowledge of the depth of the sand layer being considered, the depths and soil characteristics of the overlying strata, and the height of the groundwater table, the cyclic stress ratio in the sand layer resulting from a known maximum surface acceleration can be derived using the formula and the chart presented above.

To determine whether this calculated cyclic stress ratio will cause a soil to liquefy, Seed et al. (106) compiled a comprehensive set of data describing site conditions at locations where liquefaction did or did not occur during past earthquakes. They developed, on an empirical basis, a correlation between the penetration resistance of the soil (corrected to an effective overburden pressure of one ton per square foot), N_1, and the cyclic stress ratio that will cause liquefaction. This correlation is shown on Figure V-5. The corrected penetration resistance value N can be calculated through the following equation:

$$N_1 = N\left(1-1.25 \log \frac{\sigma_o'}{\sigma_1}\right)$$

where N is the measured penetration resistance and σ_1 is equal to one ton per square foot, and σ_o' is the effective overburden pressure as previously defined.

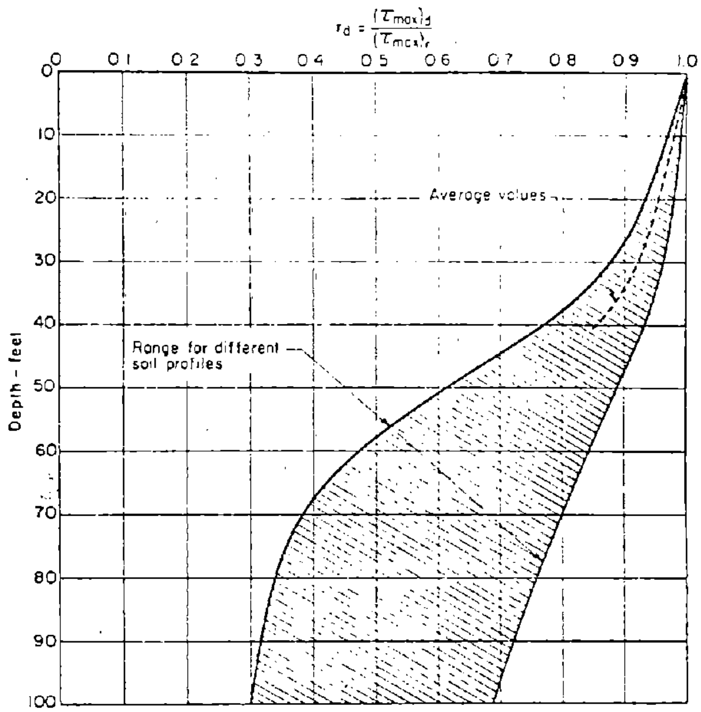


Figure V-4. Range of values of r_d for different soil profiles (112).

Thus, with a knowledge of the penetration resistance of the soil and the effective overburden pressure (calculated earlier), N_1 can be calculated for the sand deposit being considered. Combined with the calculated value of the cyclic stress ratio in that soil deposit resulting from a particular earthquake, one can use the graph in Figure V-5 to make a rough determination of whether or not the soil will liquefy.

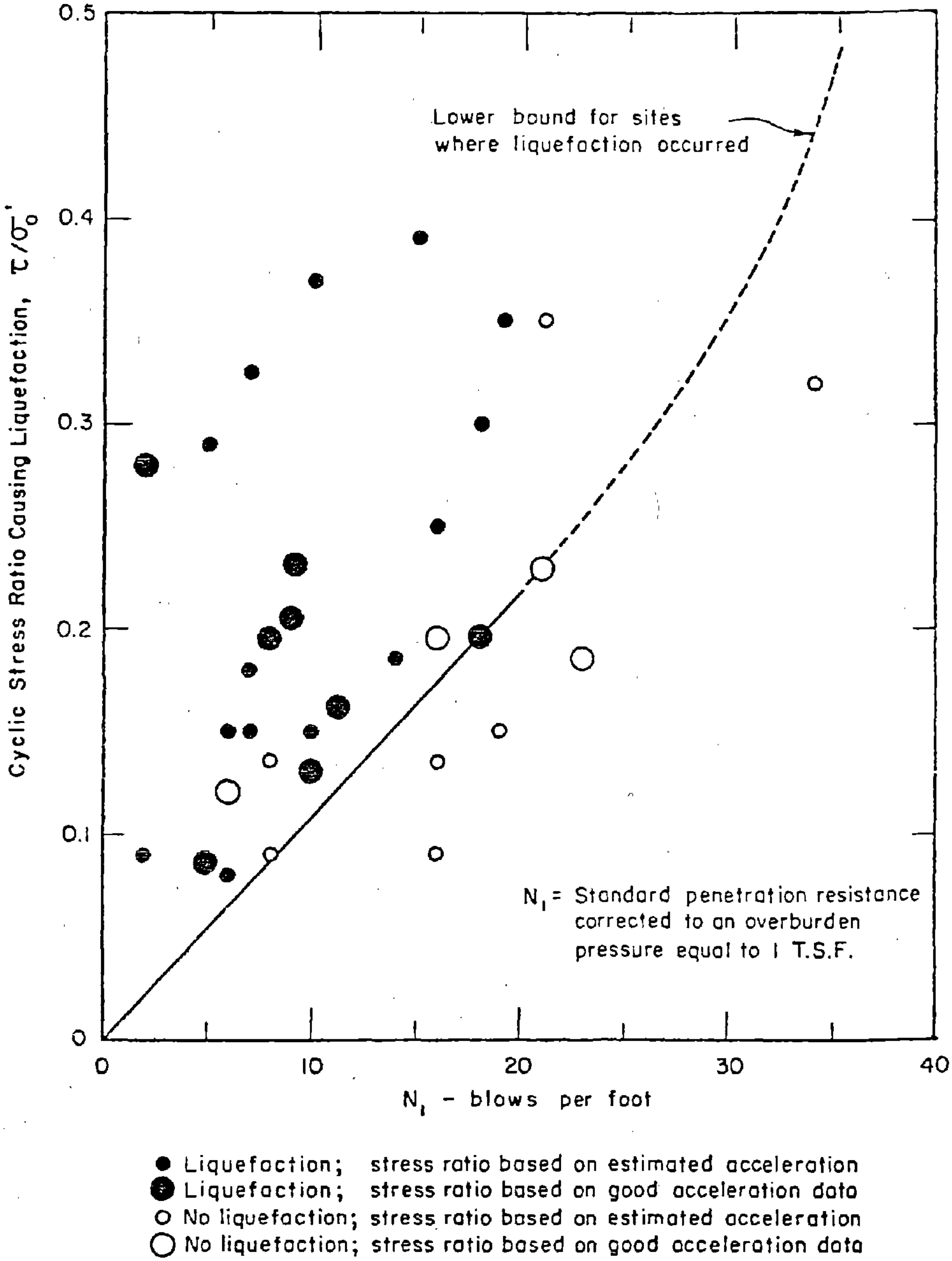


Figure V-5. Correlation between stress ratio causing liquefaction in the field and penetration resistance of sand (106).

It is generally agreed that this method is useful only for preliminary evaluations of liquefaction potential. The method does not account for significant factors such as the duration of shaking, and it depends on the questionable reliability of field measurements of penetration resistance (106). If the graph shows that liquefaction may occur, one of the following methods can be used to make a more reliable determination.

The cyclic stresses induced in a soil deposit by a given earthquake can be computed using a ground response analysis. This involves three basic steps (109):

- Assess the magnitude of ground motions likely to develop in the base rock at the site being considered throughout the period of the earthquake.
- Assuming that deformations of overlying soils are caused by vertical propagation of shear waves due to base motions, compute the shear stresses at different depths and their variation with time.
- Convert the stress history into an equivalent number of uniform stress cycles and determine the equivalent uniform cyclic shear stress developed at each level.

The end product of the ground response analysis will be a plot showing, for an earthquake characterized by a given number of stress cycles, the cyclic stress that will develop at each soil depth.

To determine the cyclic shear stress causing liquefaction at each soil depth during an earthquake characterized by the same number of stress cycles, simple shear tests or cyclic load triaxial compression tests can be performed in the laboratory under conditions simulating field conditions.

A detailed description of the ground response analysis can be found in reference 115. The interpretation of simple shear or triaxial compression test data is discussed in reference 116. Detailed discussions regarding the validity of laboratory test procedures can be found in reference 106.

RECOMMENDATIONS

The 1976 National Interim Primary Drinking Water Regulations (117) state that any new facility or any extension of an existing facility should not be located at a site which is subject to a significant risk from earthquakes. It can safely be stated that any structure located directly astride a fault will have a very high potential for severe damage if movement occurs along the fault; this should obviously be avoided whenever possible. Although it is relatively easy to locate individual structures away from fault zones, it may be more difficult in the case of water and sewer pipelines and canals. Aqueducts and pipelines crossing fault zones can be designed to minimize structural damage from fault displacements; this will be addressed in the next chapter. Another approach is to accept the probability of failure and make provisions for rapid repair. When siting aqueducts and pipelines, considerations should also be given to vertical land displacements which may occur in the vicinity of a fault. Vertical land movements may change the hydraulic gradient and render large sections of the conduit useless (038).

Siting facilities on vulnerable soils (discussed previously) should obviously be avoided whenever possible. Of course, the vulnerability of a facility to ground failure must be weighed against other factors. If the facility must be located in vulnerable areas, it can be designed to resist damage, or the vulnerability can be considered an acceptable risk. Unfortunately, sewage treatment plants and pumping stations are often sited on low lying alluvial plains subject to liquefaction. Water treatment plants supplied with water from rivers may also be located on vulnerable soils. Although facilities are rarely located on steep slopes, caution should be exercised in siting a facility adjacent to an area prone to landslides. Potential damage from tsunamis must also be considered.

SEISMIC AND GEOTECHNICAL INVESTIGATIONS

A preliminary investigation of potential sites should include a review of the seismic history of the region, establishing the relationship of the site to known faults. Technical literature such as that published by the U.S. Geological Survey can provide useful information such as the historical occurrence of major earthquakes, magnitudes near the potential site, the location of fault traces and evidence of regional fault strain and creep (101).

In regions such as southern California which have historically experienced earthquakes with relatively shallow foci, aerial photography such as low-altitude, low-sun-angle photography can be effective for identifying small-scale features related to ground deformation and faulting. Trenching has also been used in these regions for fault location when siting critical structures (118).

Areas of potential ground failure can be located by performing detailed soils investigations. Preliminary information can be obtained from soil surveys conducted by the U.S. Department of Agriculture, available through the Soil Conservation Service (119). The Earthquake Engineering Research Institute (EERI) recommends that the general geologic information presented in Table

V-1 be compiled by geologists during local investigations after earthquakes. At sites of special interest, the same source recommends that the more detailed information presented in Table V-2 be collected.

In the East Bay Municipal Utilities District (EBMUD), seismic overlay maps have been prepared to show critical information in relation to water system facilities (120). Overlays on top of basic topographic maps show the fault zones, the definite or probable locations of fault lines, the relative direction of movement on each side of the fault, the location of cracks, upthrown and downthrown sides of faults, and information sources on each fault. Areas of artificial fill, former tidal flats, and old concealed streambeds are also shown. Overlay maps such as these can be highly valuable during planning efforts.

TABLE V-1. GENERAL GEOLOGIC INFORMATION*
(014)

Report by:________________________ Address:____________________________

Occupation:_______________________ Home or business phone:_______________

Date of report:____

REGIONAL GEOMORPHOLOGY:

Description of geomorphic province in which earthquake occurred:
Major geomorphic features and lineations (relationship to rock type):

____

Topographic map(s):____

Stream patterns: ____

Relief: Maximum _________________(m) Average _______________(m)

Average slope inclinations:______________________________________

Locations of specific sites:_____________________________________

____

REGIONAL GEOLOGY

Description:

Regional tectonic setting (including tilting, warping, depression, uplift, etc.)

Regional fault system (importance of causative fault in region system)
Types of faults, tectonic relationships (maps and cross-sections)

Major rock types and their distribution (geologic map)

Volcanic activity

Ground water:

Nature, (free, confined, perched, etc.):__________________________

Map of ground-water levels based on depth to ground water

Water levels in wells (note changes): elevation:_________________

depth: _________________

location: _________________

EARTH MATERIALS:

Geologic map of area:

Cross-section delineating distribution of earth materials and geologic structures (at least one through the focus, if possible)

Complete description of earth materials (include comments on geologic age, type of material, composition [%], texture [% grain size if applicable], consolidation, moisture content, porosity, permeability, cementation, structure, origin, etc.); especially note type and distribution of Quaternary sediments

* Recommended by EERI

TABLE V-2. DESCRIPTION OF LOCAL "SITE" GEOLOGY[1] *

(014)

Report by: ______________________ Address: ______________________

Occupation: ______________________ Home or business phone: ____________

Date of report: ____

Location of Site: ____________ Latitude ________ Longitude ____________

Important landmarks in relation to site: ______________________

Street address: ______________________________________

City: ____________ State: ______________ Country: ____________

Township: __________ Range: ______________ Section: ____________

If under water, note depth: ______________________________ (m)

Current velocity: ______________________________ (m/sec)

Direction: ______________________________

Wave height: ______________________________ (m)

Distance and direction to causative fault: ____________________ (km)

Distance and direction to fault rupture: ____________________ (km)

Distance to epicenter: ______________________________ (km)

Dimensions of site: ____________ (m) x ____________________ (m)

Types of Engineering Structures on the Site, If Any:

Date of design: ____________________ of construction ______________

Building code in force: ______________________________________

Instrument location on site or near site (type): ____________________

Maximum acceleration (structure, basement or free field): ____________ g

Repeated high acceleration (general level): ______________________ g

Duration of strong shaking (X0.05 g): __________________________ secs

Sketch site, with structure(s) location, on back of sheet.

Very brief description of damage and reference complete damage report.

Earth Materials (type, age, thickness, depth below surface, density, degree of consolidation, relative density, cementation, size of clastic material, moisture content, etc.):

TABLE V-2 (continued)

Artificial fill (how constructed, age, type of compactive effort, applicable building codes)

Regolith (soil type, grain size, sorting, relative density) Holocene sediments
Pleistocene sediments
Bedrock (Tertiary or older sedimentary rock)
Seismic bedrock (if refraction survey data available)
Basement complex (dense, crystalline igneous or metamorphic rock)

Describe: depth: __________(m)

Degree of weathering: ____

Water Table Information:
Depth to water table:

Perched: ____________________________(m)

Confined: ___________________________(m)

Unconfined: _________________________(m)

Post-earthquake variations in water table: ______________________(m)

Description of grading sites, including slopes, cut or fill, height, slope angle, orientation of slope (N, S, E, W), available geology and soils reports, code in effect at time of grading, enforcement of code? Date site graded.

Draw geologic cross-section through site, down to basement complex, if possible. At least two sections, perpendicular to each other.

Geomorphology of site. Describe relation to larger area.

---

[1] To be used only at sites of special interest. Appropriate checklists of primary and secondary effects should be filled out for each site described.

* recommended by EERI

B. CONCEPTUAL DESIGN OF SPECIFIC FACILITIES

Water and wastewater facilities should be able to meet operating goals during routine operating activities and maintain the highest possible degree of functional capability in the face of unexpected occurrences. Facilities in all regions should therefore be designed with redundancy, operational flexibility and other reliability considerations. Many of these design features are relevant to seismic protection as well as other emergencies. Most conceptual design factors discussed in this section are sound practices to prepare for all emergencies with a few relating specifically to earthquakes. More detailed design criteria specifically relating to earthquakes are presented in another chapter of this report.

Information presented is based on the following major sources:

- Recommended Standards for Water Works, 1976 Edition, a report of the Great Lakes - Upper Mississippi River Board of State Sanitary Engineers (121)
- Design Criteria for Mechanical, Electric and Fluid System and Component Reliability, U.S. EPA (122)
- Earthquake planning strategies of the East Bay Municipal Utility District (EBMUD), California
- Post-earthquake reconnaissance reports

As the titles suggest, the first two sources were not specifically intended for seismic design.

WATER SUPPLY AND STORAGE

Because the continuous availability of water is basic to the operation of any water system, providing adequate supply and storage is of utmost importance. For general planning purposes, water quantity should be adequate to meet the projected water demand of the service area based on the extreme drought of record (121). For earthquake planning, the Los Angeles County Earthquake Commission recommends that sufficient surface and groundwater storage be available to provide at least a two-week supply for minimum needs (045). EBMUD has established a planning criterion to store 87 days' supply in five terminal reservoirs; this figure was based on the time required to repair a major failure in the Mokelumne Aqueduct System, the major raw water transmission system feeding the EBMUD system (120).

A key precaution for protecting the water supply in the event of an earthquake is the provision of storage downstream from a fault zone. The California Aqueduct, for example, has terminal reservoirs providing 60 days of emergency storage downstream from the San Andreas Fault (123).

The availability of stored raw water was found to be crucial during the emergency period following the 1971 earthquake in the San Fernando Valley. Had it not been for the storage in the Lower Van Norman Reservoir, portions of the San Fernando Valley would have been without water for drinking and fire protection. The reservoir continued to provide water for 17 days after the earthquake while it was being drained (124).

Adequate storage of finished water is also important. EBMUD, for example, has established a planning objective to size distribution system storage reservoirs at two times maximum day demand in pumped zones and 1.5 times maximum day demand in gravity fed zones (120). The East Bay district also recommends that distribution reservoirs be operated to allow for drawdown of no more than

30 percent capacity under normal operating conditions (120). If possible, stored finished water should be located as close to the consumer as possible to minimize the exposure of the finished water transmission pipelines.

Redundancy of water sources is also of primary importance. Redundancy of water sources can be achieved through the use of multiple sources (e.g., a proper balance of supply from surface reservoirs and groundwater basins), imported water from aqueducts, and interconnections with neighboring water systems (125, 126).

The importance of maintaining more than one water source was clearly illustrated during the San Fernando earthquake of 1971. At the time of the earthquake, water for the city of San Fernando was supplied solely from wells; their failure and contamination resulted in severe problems in emergency supply. Alternate sources of water supply to the city of Los Angeles, on the other hand, and the flexibility of its back-up facilities, resulted in the rapid restoration of normal service.

A number of recommendations have been made concerning general design concepts for reservoirs and wells. EBMUD has established a planning objective to provide the capability to drain or substantially reduce the level of water in embankment type distribution reservoirs within 36 hours or less (120). The importance of dewatering facilities was illustrated during the 1971 San Fernando Valley earthquake. Substantial damage to the Lower Van Norman Reservoir dam necessitated rapid drawdown to prevent failure of the dam. The benefits of providing telemetering and remote-control facilities was also demonstrated during this earthquake; system adjustments needed to increase the outflow were facilitated by the telemetered flow data available to water-control engineers at LADWP's general office building in Los Angeles (039).

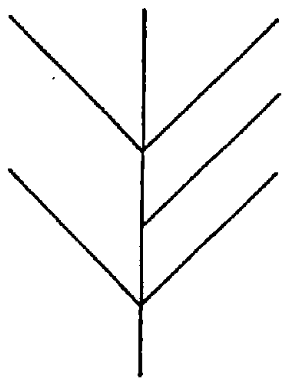
When wells are used to supply water, sound practice includes protecting the site from potential contamination through ownership, zoning, easements, leasing, or other means such as fencing (121). The Great Lakes-Upper Mississippi River Board of State Sanitary Engineers also recommends that the total developed ground water source capacity equal or exceed the design average day demand with the largest producing well out of service (121). Again, these considerations are not specific to seismic design, but were developed to deal with all emergency situations.

TRANSMISSION AND DISTRIBUTION LINES AND SEWERS

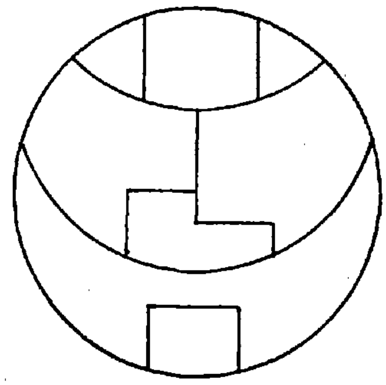
Because transmission, distribution and sewer lines of necessity cover large areas, they are particularly vulnerable to earthquake damage. For this reason, system design must provide for redundancy, operational flexibility, and rapid inspection and repair capability.

The need for redundancy in transmission lines is obvious. If two or three alternative routes are available for conveying water to a community, then damage rendering one of these routes inoperable will not cut off all water from that community. In a redundant distribution network, water can be rerouted around damaged areas, restoring service to all customers but those directly served by the damaged pipe section.

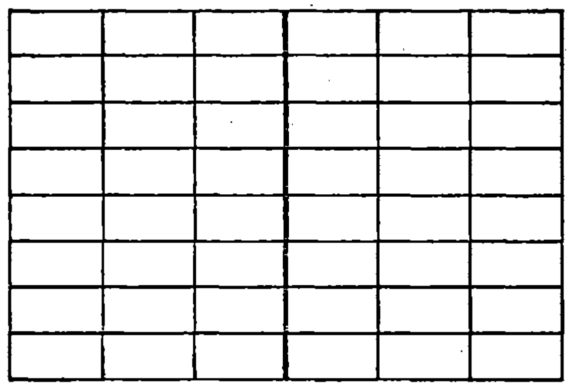
Redundancy in a distribution network can be measured by the number of paths available from the treatment facility to a given location. Figure V-6 illustrates some variations in distribution systems. Older systems and systems in sparsely populated areas may have feeder lines with branches, as shown in V-6 a, with the principal feeders arranged in a circular loop system such as that illustrated in V-6 b. High value districts or business districts normally have grid systems similar to those shown in V-6 c and V-6 d. The grid pattern with looped feeder lines shown in V-6 d provides



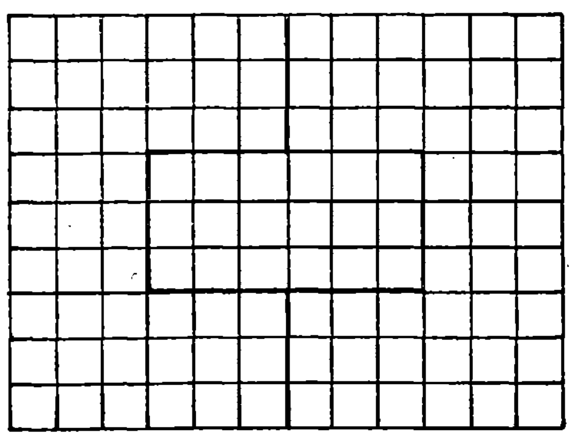
(a.) Branch system



(b.) Circular loop network



(c.) Grid network with a single transmission feed



(d.) Grid network with looped transmission feed

FIGURE V-6. Distribution network variations (after 119) the highest level of redundancy normally found in a distribution network (119). Redundancy in water distribution systems can also be increased significantly if the system is connected to two or more finished water sources. The ability to isolate damaged pipelines and to reroute water depends on the strategic placement of shut-off valves throughout the distribution system. The ready isolation of pipeline breaks will minimize local erosion damage, save stored water for subsequent emergency use (127), and allow the less damaged parts of the system to be kept under pressure (037). When locating shutoff valves, particular attention should be paid to areas of likely damage, such as faults or poor soil conditions. The LADWP recommends that shutoff valves be placed on either side of a pipeline crossing a fault (477). Up-to-date maps showing mains and shutoff valves are mandatory to control the water flow in the event of a pipe rupture (128).

Seismic design of transmission and distribution systems should also include the following considerations:

- Diversion systems should be provided to accommodate runoff and flooding resulting from a damaged aqueduct, or, if possible, to store escaping water (038, 045).
- Where a pipeline crosses a fault zone, the use of more than one smaller pipes in parallel rather than a single large pipe will increase the possibility of some flow being maintained (123).
- Standardization of pipe sizes at fault crossings and other vulnerable areas, combined with the maintenance of an adequate stock of repair piping, will expedite repair (123).

- The system should be designed to readily accommodate the installation of temporary pipelines above ground, parallel to the damaged lines (123).

With regard to sewer lines, it has been recommended that double lines of smaller pipe be used in place of larger single lines in critical locations. The overriding sewer design factor, however, is usually the capacity of pipe needed to meet existing or future demands (033). The Los Angeles County Earthquake Commission recommends that for public health and safety, emergency overflow lines to the storm water drainage system be considered at strategic locations, especially for large trunk sewers (045). This, however, may contradict some federal and state policies.

To facilitate operation under post-earthquake conditions, the provision of telemetering and remote control facilities in water systems has been recommended so that information on the system's status is immediately available and changes in system operation can be readily implemented (123). Although remote control capability is especially important when road passage may be impassible during an earthquake, relying entirely on remote operation is risky because power outages and disruption of telephone lines may disrupt communications. For example, the Chatsworth High Line needle valve and the Maclay High Line radial gate lost their remote control capability for hours after the 1971 San Fernando earthquake because of power outages. For this reason, it is also necessary to provide for manual operation of valves (039).

Opinions differ as to the desirability of providing automatic shutoff valves to prevent excessive water loss from an extensively damaged area. Having a valve automatically close due to a main break may cut off fire fighting supplies at a critical time (129). If automatic shutoff valves are installed, pressure-activated valves are probably more suitable to seismic design than electrically-operated valves, for the latter would be rendered inoperable in the event of a power failure. Again, manual overrides should be designed into the system.

A major consideration in planning the system layout is maintaining potable water separate from wastewater and other potential contaminants. In the event of an earthquake, wastewater escaping from broken sewers, cesspools and septic tanks can enter wells and distribution lines through cracks. For example, most of the wells serving the city of San Fernando became contaminated from the above sources during the earthquake of 1971 (123). After that earthquake, it was recommended that all septic tanks and cesspools within a specified distance of a well be abandoned, cleaned, and backfilled. If no other methods of sewage disposal were available, it was recommended that the well be abandoned(033).

The Los Angeles County Earthquake Commission recommends that water lines be installed above sewer and petroleum lines (045). Guidelines for separating water mains and sewers are presented in <u>Recommended Standards for Water Works</u> (121); these have not been specifically developed for seismic design, but rather for general good practice. For water mains located parallel to sewers, this source suggests that the mains be laid at least 10 feet horizontally from any existing or proposed sewer. If this is impractical, the main may be laid in a separate trench or on undisturbed earth shelf provided that the bottom of the main is 18 inches or more above the top of the sewer. Where water mains cross sewers, the provision of a vertical distance of at least 18 inches between the respective pipes is recommended.

WATER AND WASTEWATER TREATMENT PLANTS, PUMP STATIONS AND POWER SUPPLY

Seismic planning considerations for treatment plant, pump station and power supply are similar to those for general system reliability, both requiring operational flexibility and backup capability. Designing for system reliability requires that operations remain uninterrupted during routine maintenance activities and therefore requires backup capability for a wide range of unit operations. Seismic planning, on the other hand, is primarily aimed at providing disinfected water and permitting rapid repair of essential systems. For this reason, the list of required backup systems for basic emergency operations is more limited and generally depends on a unit's functional analysis rating (see Chapter III). Furthermore, the seismic requirement of providing adequate firefighting capability during the first 24 hours after an earthquake calls for supplemental design considerations such as complete bypass capability. Both seismic and non-seismic system design are discussed in this section.

Recommended non-seismic water and wastewater treatment plant design criteria are presented, respectively, in Recommended Standards for Water Works (121) and Design Criteria for Mechanical, Electric, and Fluid System and Component Reliability (122). Both sources emphasize the need for system backup and bypass capability, as well as auxiliary power sources. Various degrees of backup capability in water treatment plants are required for rapid mix, flocculation and sedimentation units, gravity and pressure filters, pumping units, and chemical equipment (121). Backup capability or redundancy is required in most wastewater treatment plants for bar screens, pumps, primary and final sedimentation basins, trickling filters and activated sludge basins, blowers and mechanical aerators, chemical flash mixing and flocculation basins, and disinfectant contact basins (122). These backup requirements are based on potential failure of individual equipment items and the need to maintain non-seismic operational goals. In both water and sewage plants, standby chlorination equipment is particularly important after an earthquake to provide disinfected water and prevent the transmission of disease.

Bypass capability in water and wastewater treatment plants is typically required for many unit operations (121, 122). In the event of an earthquake, bypass capability through the entire plant is particularly important. Maintaining hydraulic flow through a water treatment plant would insure the availability of water for firefighting or, if chlorinated, for domestic use. Maintaining flow through a wastewater treatment plant is necessary to prevent backing up of flow in the sewer system; chlorination should be provided if possible.

Essential features of a water treatment plant bypass include adequate valving to isolate the connecting pipe section, bypass connections in a well-drained pit, an inspection port normally left open with a closure plate stored in a separate location until the bypass connection is necessary, and a separate source for chlorination of the bypassed water (123). EBMUD has either installed a separate chlorinator or extended a solution line from existing plant chlorination at its bypasses (039). The determination of the acceptability of bypassed water for human consumption must be left to public health officials.

Bypassing water through a treatment plant is usually not encouraged, and is often not permitted, by many regulatory agencies. To a lesser extent, the same is true for wastewater bypassing. The decision to incorporate bypass capability into either type of plant, then, must be arrived at jointly by the individual plant management and the appropriate public health agency. The decision to allow installation of a bypass will probably depend on developing a method for insuring that the bypass is utilized only during extreme emergency conditions.

Multiple treatment plant supply and discharge lines have been recommended for seismic resistant design. This, again, supplies redundant facilities (120, 126).

Water and wastewater treatment plants should be designed to facilitate repair. Drains and pumps should be sized to permit rapid dewatering of the units, and chemical feed equipment and lines should be accessible for inspection and repair. In-line valves to isolate main wastewater pumps should be provided (122).

The U.S. EPA recommends that emergency planning provide for the installation of emergency alarm systems in chlorine buildings at wastewater treatment plants. The alarm, actuated by pressure differentials resulting from equipment breakdown in the chlorine distribution system, would automatically start exhaust fans (130). Alarms and annunciators should be considered to monitor the condition of all pieces of critical equipment at water and wastewater treatment facilities.

Maintaining power is vital to the operation of water and wastewater treatment facilities. Redundancy in power supply should be provided to treatment plants and pumping stations through two separate lines, each from an independent utility substation. At least one of the power sources should be a preferred source, i.e., a utility source which is one of the last to lose power from the utility grid due to loss of generating capacity (130). Although this arrangement would provide for the maintenance of power in the event of failure of one of the substations or lines, it is not unlikely that both may be rendered inoperable during an earthquake. The water or sewer system owner generally will have no direct control over this type of power system to control seismic design. An on-site auxiliary power system also eliminates the possible weak link of the power transmission system. For this reason, on site auxiliary power systems should be provided as the secondary power source at critical facilities in earthquake-prone regions to maintain power in the event of a commercial power failure.

It is very important that the backup power system be given a very high level of seismic protection. Often, this is overlooked in the planning and design phases. Experience has shown that where backup power supplies are not protected, they will fail as readily, if not more so, than the main power system.

The Los Angeles County Earthquake Commission recommends that all pumping stations have an emergency power source, such as an internal-combustion engine, to run at least one pump large enough to provide a minimum water supply and/or fire protection (045). Diesel-powered generators that automatically come on-line when the normal power supply is interrupted are often used in many utilities (120). Portable electric generators can be used at pumping plants with adequate wet well storage where intermittent operation may be adequate, with terminals provided for quick connections (124). Portable generating units can also be used for emergency power for dispersed pumping plants serving trunk sewers (092).

As an alternative to diesel or LPG engines at key booster stations, the Crestline Village County Water District and the Crestline-Lake Arrowhead Water Agency in California have each installed gas engine powered booster pumps on trailers. Fittings and connections permitting rapid hookup of the pumps have been constructed at certain pressure zones. These pumps could be shared among several different utilities if standard dimensions and fittings are used (123). Hydrants connected to pipelines on either side of pumping stations will allow fire-truck pumpers to be easily connected, bypassing the pump station.

High and low pressure shutoff switches should be considered for all pump stations. If a critical valve in a water distribution system is inadvertently closed following an earthquake, the pump could be pumping against a dead-end system causing pump overheating and failure. A high pressure cutoff switch would alleviate the problem. If an earthquake has caused pipe failure in the system on the pump discharge side, pumped water could be wasted. A low pressure shutoff could prevent this (120). It may also prevent the pump from running dry if waterlines are damaged on the suction side of the pump.

C. SEISMIC RISK ANALYSIS AND LIFELINE NETWORK SYSTEM RELIABILITY

A number of researchers have developed procedures for assessing seismic risk to lifeline network systems (131, 132, 133, 134, 135, 047, 136, 137, 138, 139). The intent of these analyses has been to optimize the lifeline system design by minimizing capital and operation and maintenance costs while maximizing system availability after seismic events. Analyses have also been considered for calculation of earthquake insurance premiums and disaster mitigation planning.

Considerable work has been done in recent years to develop various approaches to seismic risk analysis for pipeline systems. While it is evident that the state of the art is advancing in this area, the lack of input-damage data and the need for significant assumptions have weakened the procedures developed to date. Emphasis in past earthquake reconnaissance studies has been placed primarily on above-ground structures (e.g., buildings, bridges, towers). Most earthquake investigators have not been familiar with water and waste systems, so that damage in these systems often goes underreported. Also, because pipelines are usually buried, the complete identification and characterization of damages often resumes several months after the earthquake in contrast to readily observable above-ground structures. This lack of data, which has only been recognized in the last several years, requires the use of very general seismic input: pipeline damage assumptions. As more information is obtained from actual earthquake events, the utility of these approaches will, no doubt, increase. This section presents a general summary of work to date and is intended only as a background description of the subject area. The reader is guided to specific references if more detail is desired.

Risk analysis and lifeline reliability may consist of a number of independent tasks, as follows:

a. Define the earthquake events to which the area may be subjected (i.e., location, magnitude, probability).

b. Estimate the direct potential earthquake effects on a local level and their probability of occurring and/or magnitude (e.g., liquefaction, faulting, ground shaking).

c. Estimate the consequences of the earthquake effects on individual components in the lifeline system (e.g., pipeline breaks/mile).

d. Determine the effect of the combination of earthquake induced component failures on the entire system in relation to the probability of failure.

If these task areas are carried out for a number of possible earthquakes distributed in accordance with their probability of occurrence, a summation of the results will constitute the seismic vulnerability of the system.

The determination of what earthquakes the area may encounter and the probability of them occurring may be approached in several ways. Algermissen has developed earthquake risk maps for the entire country, variations of which have been adopted by the Uniform Building Code (140) and the Applied Technology Council (141) (See Figures VII-1,2,3 and 4, pages VII-15-18). These maps show pseudo-acceleration and pseudo-velocity contour lines with a specific earthquake occurrence probability for the levels of motion indicated. On the local or microzonation level, Cornell developed a method for determining seismic risk at a single site considering a spatial distribution of earthquake sources (134). Panoussis (134) extended this method to determine the seismic risk of a lifeline system geographically spread out. Attenuation of the motion with distance away from the fault was considered as well as the relative proximity of two adjacent areas being considered. Taleb-Agha developed a procedure to reduce the required computer time, a problem for larger systems (135).

Once the probable seismic motions to which an area may be subjected are determined, the direct effect they will have on the area must be determined. Shinozuka (133) has presented a means of approximating fault displacement. A number of researchers have developed methods for calculating the free field strains from seismic wave propagation as discussed in Chapter IV. Soil conditions must be taken into account. The relationship of ground failure liquefaction to earthquake intensity was discussed previously in this chapter.

A relationship of direct earthquake effects to system component damage must then be applied. Erel et al. (142) applied historical data on pipeline failure/mile from a number of earthquakes. This failure rate data was expressed as a function of ground acceleration. Shinozuka (133) has made approximations of failure from shaking rates from the calculated free field strains. Pikul et al. (143) have developed a qualitative assessment of pipe failure based on calculated free field strains quantitatively considering soil types. Hein et al. (047) approximated pipeline failure rates from liquefaction. Shinozuka (133) developed a quantitative evaluation of pipeline failure from fault movements based on procedures presented by Newmark and Hall (144) and from liquefaction based on calculations developed by Kennedy et al (043). Treatment plant damage in terms of repair cost (percent of replacement cost) was related to ground acceleration by Erel et al. (142) from information gathered by Whitman on concrete building damage. The resulting estimated damages have been used by others to approximate the direct cost of earthquake damages. A "damage probability matrix", is then developed which defines the probability of failure of a system component in a certain mode (liquefaction, faulting, etc.) given the earthquake intensity and specific soil conditions. Shinozuka (133) arrives at these damage probability matrices mathematically. Panoussis (134) suggests they can be developed by "statistics, engineering analysis and/or experience." The damage probability matrix is then applied to each system component.

The last step is to evaluate the effect of the earthquake on the overall system. Shinozuka (133) has used a Monte Carlo method to simulate various damage states for a single earthquake. The effect of a particular earthquake on the system can then be analyzed using a flow network analysis, such as Hardy-Cross. Panoussis (134) with developments by Taleb-Agha (135) developed a method to simplify a complex lifeline network into a number of parallel lines. The system consequences could then readily be derived using this approximation. A large number of earthquakes can then be simulated. A summation of the results, with simulated earthquake intensities, results weighted in accordance with their probability of occurrence, will show the probability of a lifeline system being functional following an earthquake.

Erel et al. (142) have developed a method to evaluate indirect losses from lifeline outages accounting for inconvenience, fire losses, and economic losses. Whitman et al.(145) estimated a 2% change of a least minor seismic damage taking place in Boston, indicating that projections of magnitude of the seismic problem are possible. USGS has studied earthquake losses from possible earthquakes in several areas including Puget Sound (079) and Salt Lake City (146). Seismic risk analyses were carried out on a California

State Water Project where more than 100 alignment variations were studied to minimize seismic vulnerability (147, 148). The proven data relating a given earthquake to pipeline damage with all variables known are scarce. Past studies have made gross assumptions about many of the relationships or have simplified the analyses by considering limited direct earthquake effects.

CHAPTER VI

DESIGN CONSIDERATIONS

Two basic types of design information are presented in this report. Suggested criteria for the calculation of earthquake induced forces on major equipment configurations are presented in Chapter VII. This chapter presents design considerations for specific equipment types. Some concepts are presented based on structural analysis, while others are offered as suggested techniques of good practice. The qualitative design considerations are derived from lessons learned from past earthquakes, analyses performed by large utilities and techniques developed to protect water and sewage systems during general emergencies, including floods, fires, strikes, etc. The intent of this chapter is to provide system planners and designers with alternatives to minimize the impacts of seismic activities.

The major sources of background information for this chapter were discussions with personnel from numerous water and sewage systems, design recommendations by various government agencies and professional groups, and reconnaissance reports from past earthquakes. In addition, the general technical literature was surveyed to develop concepts based on related equipment in other fields.

This chapter is subdivided into sections according to equipment categories. Water source and sewer discharge facilities are discussed, including intake structures and wells. Pipelines and channels are discussed from a practical design standpoint. A detailed analysis of treatment plant facilities follows, including concrete tankage, treatment equipment and piping. Pump stations and emergency power supplies are also discussed in this section due to their structural similarity to treatment equipment. Surface supported and elevated storage tanks are presented in a separate section. A qualitative discussion is presented on the costs of including seismic considerations in the design.

A. SOURCE/DISCHARGE FACILITIES

Potable water can be provided from either surface water or ground water sources. Surface water sources include rivers and either natural or man-made impoundments with earthen, rock, or concrete dams or embankments. Although the seismic resistant design of dams is beyond the scope of this report, intake structures will be discussed in this section. These commonly consist of tower structures used to regulate the intake of water from impoundments. Wells, the primary source of ground water, will also be discussed.

Treated sewage is generally discharged through some type of submerged outfall, often including a diffuser. These submerged structures will also be included in the present section. Submerged water intake and sewage outfall piping will be covered in relation to their foundation requirements as submerged structures.

SUBMERGED INTAKE/DISCHARGE PIPING AND STRUCTURES

This subsection includes structural design considerations for submerged piping and structure foundations.

Foundations

The stability of strata supporting intake and discharge structures and piping should be evaluated. Alluvial deposits commonly found on the bottoms of rivers and lakes may be subject to seismic induced liquefaction. Structures are commonly sited on or near slopes that may be subject to lateral displacement during an earthquake. The reader should refer to Chapter V, "System Siting, Planning and Conceptual Design", for a detailed discussion of liquefaction potential and associated soil failures.

To resist the possible effects of liquefaction, the structure may be founded on a solid stratum below that with a high liquefaction potential. Depending on the thickness of the potentially unstable layer, piles may be used, the potentially unstable material may be excavated and replaced, or the material may be stabilized in place. Refer to the Treatment Plant, Buried Tanks and Vaults section of this chapter for a detailed discussion of alternatives for foundation design.

The stability of slopes near a submerged structure is an important consideration because failure of an embankment may result in a landslide that could destroy the structure. This consideration is generally more critical where steep slopes are encountered. Seismic slope stability is beyond the scope of this report. Extensive work has been done in this area and is reported in the literature (149).

Intake Structures

Water intakes are typically columnar, reinforced concrete structures used to control the elevation and rate at which water is withdrawn from an impoundment. Structurally, these intakes are similar to chimneys, with the obvious exception of their being submerged (150). This adds a hydrodynamic effect to the earthquake response of the structure. The water inside the structure has a rigid response relative to the structure. The portion of water surrounding the structure responds with the structure, increasing the effective weight of the intake tower. This also lengthens the structure's fundamental period, which has a greater effect on the response of tall, slender structures than on short, stocky structures. This consideration is included as a design criterion in Chapter VII.

The design criteria presented in this report are to be used for the design of relatively simple structures. Tall, slender, cylindrical structures such as intake towers and chimneys require a complex structural analysis. The use of the procedures presented herein should be limited to preliminary design. The reader may refer to a report by A.K. Chopra and C.Y. Liaw (150) for a detailed analysis, a short discussion of which is presented below.

Chopra and Liaw recommend that the intake tower be designed to respond within its material's elastic range for moderate earthquakes which may occur several times during the design life of the structure. As a minimum, the structure should be designed to yield, but not collapse, when subjected to an intense earthquake.

EBMUD decided to replace the intake tower in the Lower Van Norman Reservoir destroyed in the 1971 San Fernando earthquake with a structure that would respond within the elastic range when subjected to intense shaking. The design included a cylindrical structure which at the base had 6' thick walls with four concentric rings of steel reinforcement. Each ring consisted of #11 bars, 12" O.C. vertical and #10 bars, 11" O.C. horizontal, apparently a very massive, strong installation (150).

The other option is to provide ductility in the design to absorb a portion of the energy transferred to the stucture, allowing the structure to yield but not to collapse. The difficulty with this design approach in stack-like structures is that if extensive yielding takes place in one section, it may generate large displacements in another section and cause the structure to collapse. Ductile concrete design would be used in such a design, requiring concrete under compression to be confined. For cylindrical intake structures, this would require multiple rings of reinforcement (150).

Chopra and Liaw developed a simple analysis for the preliminary design of an intake tower, this preliminary analysis accounting only for the fundamental modal response of the structure. For higher mode responses that are critical for proper design, a computerized finite element approach is included.

The design engineer may choose to design an intake structure to keep out debris generated during small earthquake-related landslides. Bar screens are a possible alternative (036).

WELLS

This subsection emphasizes well casing and well pump design. Equipment, piping and building structures associated with wells will be discussed to the extent that their particular design is related to wells. A detailed discussion of general equipment, piping and building structures is given later in this chapter.

Well casings are similar to buried pipelines (discussed in the next section) in that they respond or move with the surrounding soil during an earthquake. There are two general types of movement that may be experienced by the soil (and well casing). The soil may move along fault lines or may deform from structural failure, i.e., liquefaction and landslides. It is unreasonable to expect well casings to resist these types of major movement. The soil may respond to earthquake motions of the base rock with compression and shear waves passing through the soil. Shear waves may bend a casing by inducing relative lateral displacement of the casing as a result of the lag between the time the wave reaches two distant points on the casing. The effect of the lag time of the compression wave induces axial strain on well casings in much the same manner. A properly designed well casing should be able to accommodate these types of motions.

Siting

In most cases, water availability and quality are the primary non-seismic criteria for well siting. Beyond these, there are several considerations to avoid seismic damage to wells.

Because there is little that can be done to resist the effects of major ground deformation, wells should be located away from fault lines or traces, unstable soil conditions, and steep slopes (151). Wells should be separated from sewage facilities (i.e., sewers and septic tanks) by as great a distance as possible. Care should be taken to locate and pump out nearby abandoned septic tanks. If sewers and septic tanks are required near wells, they should be designed and constructed using a conservative seismic resistant approach.

Casing Design

The casing should be designed to resist the effects of compression and shear waves. Steel casings commonly used for wells are generally flexible enough to accommodate the bending from shear waves to which they may be subjected during an earthquake. There seems to be no reason to change the type of casing materials currently in use. Assuming that the casing moves with the surrounding soil, it should deform to the same extent, depending on the casing's rigidity (within reasonable limits). The bending stress induced on the casing is dependent on its deformation and not on the casing wall thickness. As the casing diameter increases, the bending stresses increase; casing diameters should therefore be minimized.

The amplitude of the displacement of rigid soils, i.e., dense sands or stiff clays, is less than that of softer soils. Lateral as well as axial casing displacements, and therefore induced stresses, are thus lower in more rigid soils (151). Compression waves traveling parallel to the pipe axis induce axial compression and tensile strains on the casing. Wells differ from buried pipelines in this situation in that both ends are not restrained, allowing movement and release of strains. Therefore, axial stresses are believed to increase as the casing length increases, reaching a maximum at one-half of the length of the compression wave plus some length to develop soil/pipe friction. Little information is available on vertical compression wave lengths and amplitudes, so that required design displacement is difficult to predict.

When bending and axial stresses are combined, local buckling of the casing may occur. Furthermore, local soil failure may change local stress patterns, causing buckling. Nazarian (151) has proposed a method to calculate all of the stresses in a well casing when subjected to earthquake motions. The reader may refer to his paper for a detailed stress analysis procedure. In that paper, Nazarian calculated the casing stresses on a 16"/10" diameter telescoping well 800 feet deep to be well within the allowable working stress limits of the casing material. Based on that example problem and historical well casing failures previously described in this report, it would appear that most earthquake motion induced casing failures occur from local stress concentrations and soil failures.

It may be possible to relieve some of the axial stress by providing joints along the casing that have axial flexibility. A possible means of providing this flexibility may be to design the casing such that the pipe sections "telescope" (033). The telescoping joints could be sealed using a lead packing commonly used to seal telescoping well screens. Sleeve-type joints have also been considered.

A general rule in seismic engineering that may be applicable here is to minimize structural discontinuities. In the well casing, this may apply to casing joints, standard or telescoping casing/well screen joints, and constraints at the ground surface. It is obviously impossible to avoid all discontinuities, but care should be taken in their design to minimize possible locations for stress concentrations that could cause local buckling.

As previously discussed, the well casing may move both vertically and horizontally in response to an earthquake. The well casing should be separated from the well house floor or surrounding concrete pad, with a flexible material filling the void. A one (1) inch gap has been suggested (151). The discharge pipe exiting from the well should be separated from connecting piping with a flexible joint.

Pump and Piping Design

Turbine pumps are most often used in deep wells. These pumps consist of a series of small diameter pump bowls, including impellers. Water is drawn into the first (bottom) stage and discharged through the last (top) stage, through a discharge riser pipe to the ground surface. The pump hangs from the top of the well supported by the discharge riser pipe. The pump motor is usually located immediately above the well and connected to the pump with a continuous drive shaft (no flexible joints). The drive shaft is supported with bearings inside the discharge riser pipe. This type of pump is not recommended for seismic resistance. The drive shaft must be kept perfectly straight to avoid vibrations, bearing overloads and pump failure. An earthquake may slightly bend the casing, thus bending the drive shaft. The advantage of this type of pump is that there are no electricial parts under water that may be subject to electrolysis. They have historically provided many years of service in numerous installations.

Submersible pumps are used primarily for small wells and, in special situations, in larger wells. Submersible pumps are similar to deep well turbine pumps except that the motor is submerged and close connected to the pump. The long drive shaft and the requirement for a perfectly straight well casing are therefore eliminated. This is advantageous for seismic resistant design. Even if doglegs develop in a well during an earthquake, this type of installation could remain operable. The disadvantage of submersible pumps is that their life expectancy is considerably less than that of vertical turbine pumps. The submerged electrical components are subject to electrolysis and the motors are more difficult to properly maintain.

It has been recommended that the discharge riser pipe and pump for submersible pump installations be equipped with lateral supports such as springs (151). These would allow for setting of the pipe and pump but would provide some lateral support. Nazarian (151) has proposed a method for calculating stresses in a submersible pump riser pipe. His calculations indicate that the maximum stresses in the discharge pipe are well within standard pipe material allowable stress limitations.

Earthquake vibrations may cause an increase in turbidity in the well. Depending on the nature of the installation, the pump may be fitted with a seismically activated switch that will shut the pump off in the event of an earthquake (151). This would prevent turbid water from entering the system. However, if fire protection is a primary concern, automatic shut-offs may be harmful.

Stand-by or emergency power should be provided for well installations. A detailed discussion of this factor is presented elsewhere in this report. The reader should refer to the Treatment Plant Design Considerations Section of this chapter for a discussion of other associated equipment and structures.

B. TRANSMISSION, DISTRIBUTION AND COLLECTION SYSTEMS

This section presents both general and detailed design considerations for water transmission and distribution systems and sewage collection systems. General design considerations previously discussed in Chapter V are reviewed. Design alternatives for resisting potential seismic failure modes are presented for buried pipelines, fault crossings and open channels.

GENERAL CONSIDERATIONS

A great deal of money and effort can be saved if system siting is considered during the planning stage. Faults should be avoided entirely; if this is impossible, they should be crossed in a perpendicular fashion. (Considerations for fault crossing design are presented later in this section.) Unstable soil conditions such as hillsides, embankments, and areas with high liquefaction potential should also be avoided.

A system should be designed under the assumption that some components will fail during an earthquake. Redundancy should be provided where possible;for example, dead ends should be avoided so that if a single pipeline fails, water can be rerouted through an alternate pipeline (125, 120, 152).

Valving is an important consideration when strengthening a water transmission or distribution system to withstand earthquakes. Adequate spacing and strategic location of valves are key factors so that:

- damaged portions of the network can be isolated for repair
- damaged sections can be closed off to reduce water losses

- water can be rerouted around damaged portions to maintain service to undamaged communities.

In general, a valve should be located so that it can be easily and rapidly operated, repaired and maintained (125). All side connections or appurtenances should be valved at the main. Spacing of valves should not be more than 600 feet apart in 6 and 8-inch mains, or 1,000 feet apart in 12 and 16-inch mains, so that the length of distribution piping shut down at one time can be minimized. EBMUD requires reduced valve spacing in areas of unstable ground (153).

Easy access to all parts of a system should be provided so that in the event of earthquake damage, repairs can be accomplished quickly.

System materials should be standardized to the greatest possible extent so that a minimum stockpile of materials is required.

Waterlines should be installed above sewer lines where possible to prevent contamination from damaged sewers. A minimum horizontal separation of 10 meters has also been suggested (123).

A detailed discussion of many of the general design aspects can be found in Chapter V.

BURIED PIPELINES

Buried pipelines may be subject to earthquake induced loadings from:

- strain in the ground surrounding the pipe from propagating seismic waves
- soil failure such as liquefaction or landslides
- fault movement

Pipe Strain From Seismic Wave Propagation

The magnitude of the ground strain, which is dependent on the soil stiffness, is smaller in stiffer soils. Pipelines should therefore be sited in areas of firm soil where possible.

The strain on the ground that is transferred to the pipe will be taken up by straining of the pipe or by pipe-joint relative movement. According to the result of Wang's analysis (072) and the type and extent of seismic damage to modern pipelines in recent earthquakes, it would appear that the materials and joints currently used in construction can adequately accommodate seismic wave induced pipe strains. Wang (072) calculated material strains and pipe joint displacements for pipeline materials commonly used, listed below, and found them to be within an acceptable range.

- cast iron pipe (flexible joints)
- ductile iron pipe (flexible joints)
- steel pipe (welded joints)
- reinforced concrete pipe (flexible joints)

The flexibility/ductility of the piping system is primarily attained within the steel in the steel pipe system and within the joints in the other materials. The flexible joints refer to a push-on type joint with flexible gasket material.

Wang (154) has presented a procedure for calculating material stresses on pipelines from seismic wave induced strains to be used in conjunction with standard pipe stress calculations (e.g., wheel loading, back fill, etc.) However, Wang's calculations are based with the assumption that the pipe moves with the surrounding ground with no slippage. If slippage occurs between the pipe and surrounding ground, stress may build up at pipe junctions. Therefore, flexible connections should be installed where ground wave propagation or pipe movement may not be homogenous as follows:

- bends, tees, crosses, etc.
- connections to structures (e.g., manholes, well casings, tanks, buildings, pump and meter pits, etc.).
- valves and hydrants (125)
- interfaces between dissimilar soil masses

Shinozuka has presented a method for calculating pipe strains at bends and tees (070). Details on installation of flexible connections are discussed later in this section.

Unstable Soils

Pipe movement may also be caused by liquefaction of surrounding soils. Liquefaction can be resisted by stabilization of the soil strata. Drainage of the area by the installation of gravel piles has also been suggested. This is discussed in detail in Section C of this chapter. In addition, the pipe could be installed beneath the potentially liquefiable layer (20-50' deep) or above ground supported on piles extending below the liquefaction zone. A pipeline can conceivably be designed to resist the bouyant forces exerted on it from the surrounding liquefied soil by supporting the pipeline on piles. Kennedy et al. (043) have presented a procedure to calculate the stress on a buried pipeline from liquefaction of the surrounding soil. However, in gravity flow systems, movement of a pipe by flotation could disrupt the system's operation even if the pipe is still intact.

Above-ground pipelines which must traverse hillsides should be securely anchored with girder supports to anchor blocks. In order to resist sliding, the anchor bolts can be attached to deeper bedrock with long thrust bolts.

Pipelines installed in areas of earthfill have been found to sustain damage due to soil compaction and settlement as a result of seismic shaking. Earthfill areas should be carefully and completely compacted prior to installation of the pipeline to minimize damages caused by differential settlement of soils between areas of cut and fill.

Direct tectonic movement may also cause pipeline movement. The design of pipeline fault crossings will be discussed in detail later in this chapter. However, tectonic movement may not be limited to a discrete fault crossing. Movement with grabens, hoists, etc. may take place nearby the actual fault. Pipeline flexibility and ductility will compensate for some movement.

Flexible couplings should be considered at the boundaries of areas where ground failure may occur, such as road embankments, hillsides and zones of potential liquefaction, settlement, landslides, etc.

Considerations for selecting materials to increase the ability of the pipeline to withstand seismic earth movements include:

- use of ductile iron or steel pipe instead of brittle materials such as asbestos cement or unreinforced concrete (044).
- although it has not yet received wide acceptance, use of plastic pipe in lieu of vitrified clay pipe (relatively rigid and brittle) for sewers (064, 127).

EBMUD (153) requires that metallic pipe with restrained joints be used in unstable gravel and at fault crossings. Butt welded, double welded, or restrained articulated joints are required. Steel pipe is normally used because it is less expensive than ductile iron and is quite ductile. Arc welded joints are stronger than gas welded joints (155).

Flexible Pipe Joints and Couplings

Pipe joint flexibility is an important consideration for most types of pipe (cast iron, asbestos, cement, PVC, ductile iron) when designing to resist seismic movement. Flexible pipe connections should, in fact, be used in most piping systems where unusually large movements are expected. Flexibility of pipe connections to structures should also be considered.

Push-on joints as well as mechanical joints provide flexibility by allowing axial, angular and rotational movement. Some pipe offset can be accommodated across a length of pipe with flexible joints at both ends. The flexibility is provided by the rubber gasket which maintains the seal.

Seismic wave propagation will move pipe joints relative to one another primarily axially, both in compression and tension (pulling apart). All types of joint relative movement could be encountered as a result of other types of seismic movement.

Pipe joint design should be aimed at providing adequate axial flexibility with provisions for some angular movement. Rubber gasketed bell-and-spigot push-on joints allow an angular deflection of 3-5^{o}, depending on the pipe size. They will also allow some axial movement. When the joint is installed, care should be taken to avoid pushing the joint "home" to allow for some axial expansion. A procedure for calculating the amount of axial movement in joints has been reported by Wang (072).

If standard joints are not adequate to accomodate expected movement, the joint could be modified by the pipe industry to have a longer bell which would allow greater movement. This, however, would reduce the joint's angular movement capabilities.

One method proposed to enhance the capability of a joint to withstand seismic axial movement is to "pad" the point of contact between the end of the spigot and the bell.

"Pull out" of joints has historically been a problem. Where this mode of failure is expected, such as in areas of unstable soil, the push-on joints can be restrained; this would allow some axial movement but would not allow the joints to pull apart. Soil strains transferred to the pipe would then be transferred to adjacent joints. The restrained joint is similar to a push-on joint except that a separate retainer ring is attached to both the bell and spigot end of the pipe. The rings are then loosely bolted together, allowing some movement but stopping it short of pulling apart. The joint should be covered with a polyethylene or other material to keep the "moving parts" free of debris. A restrained joint is shown in Figure VI-1.

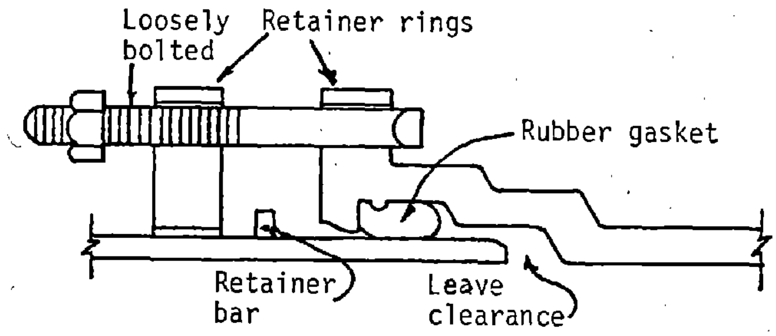


Figure VI-1. Typical restrained push-on joint to permit greater axial displacement

When additional flexibility is required, restrained expansion connections can be used (see Figure VI-2).

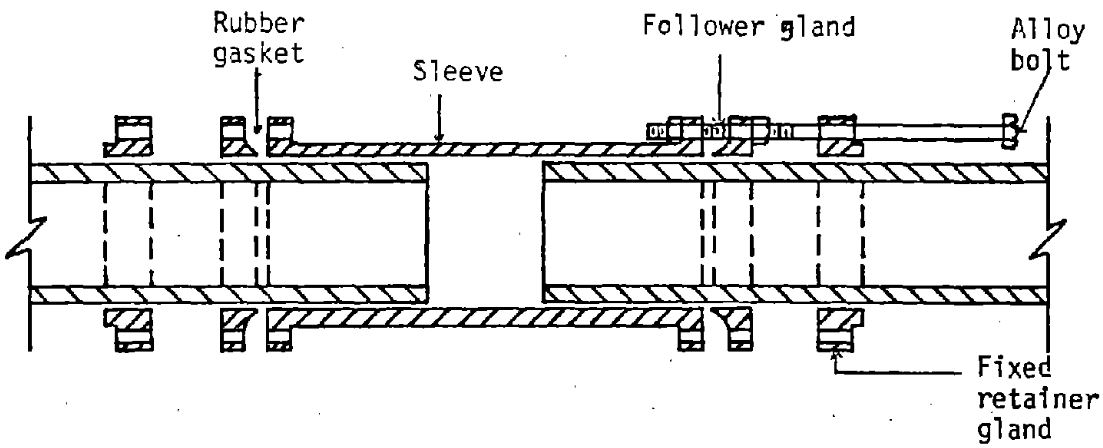


Figure VI-2. Restrained expansion connection (044).

The restrained expansion connection consists of a long solid sleeve with a mechanical joint follower gland and gasket at each end. Compression of the follower gland against the gasket builds up friction between the sleeve and the pipe. Vibrations due to seismic forces are dampened and fluid losses are eliminated by this feature. The mechanical joint retainer glands are fastened with set screws and positioned on the pipe to limit movement of each pipe end toward the midpoint of the sleeve. Outward movement of the pipes is also restricted by the fixed retainer glands. To prevent corrosion and protect the sliding surfaces from fouling, a polyethylene encasement should be placed around the assembly prior to backfilling (044). This type of joint is commercially available.

Piping connections to structures such as manholes should also be flexible. A flanged sleeve of high quality rubber can be cast into the wall of the manhole. The sleeve protrudes from the base of the wall at a right angle and is slipped over the end of the sewer (see Figure VI-3). The sleeve is secured around the pipe with a strap clamp. The rubber material of the sleeve should be resistant to raw sewage, ozone, acids, etc. Other designs are also available to accomplish the same result.

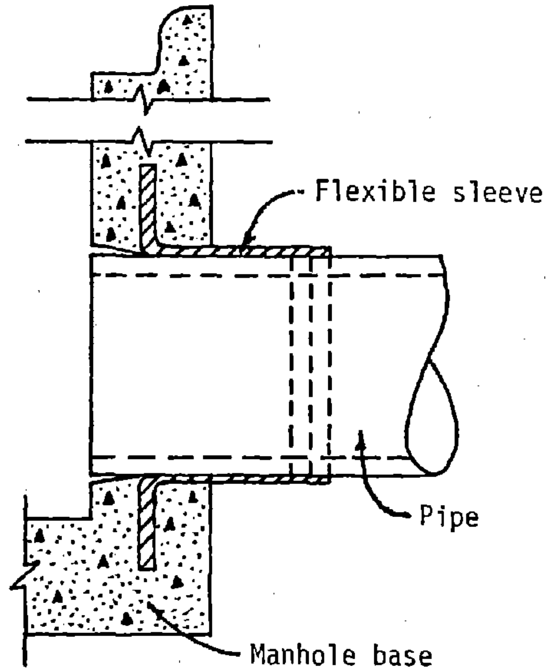
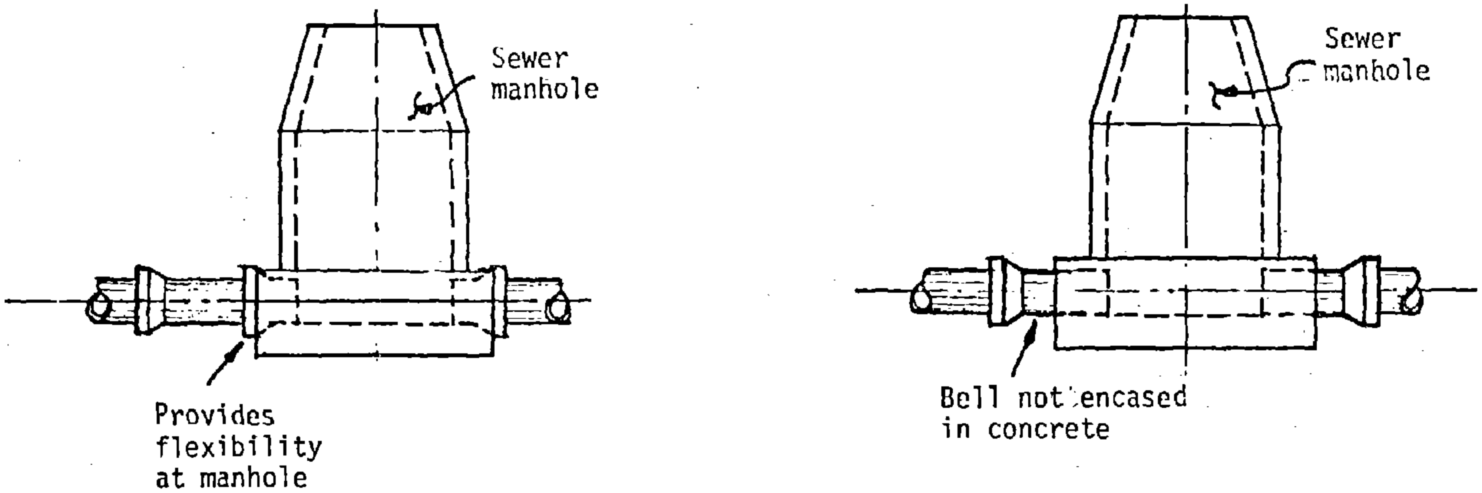


Figure VI-3. Cross section of manhole sleeve (156).

Other types of flexible connections have been suggested such as ball and socket joints, metal bellows, and rubber bellows. These types of joints are normally used above ground and are discussed in detail later in this chapter.

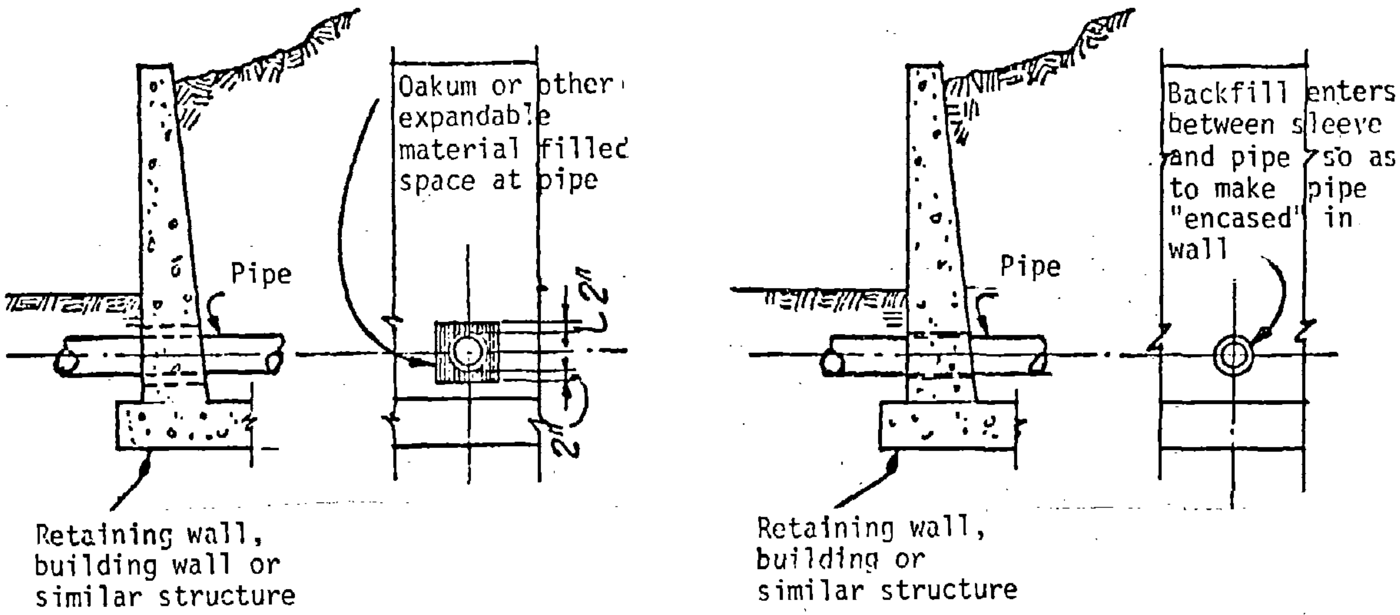
A number of details showing proper and improper installation of flexible connections are shown in Figures VI-4 to Figure VI-9.



(a) Good practice

(b) Poor practice

Figure VI-4. Manhole pipe connections (157, 158).

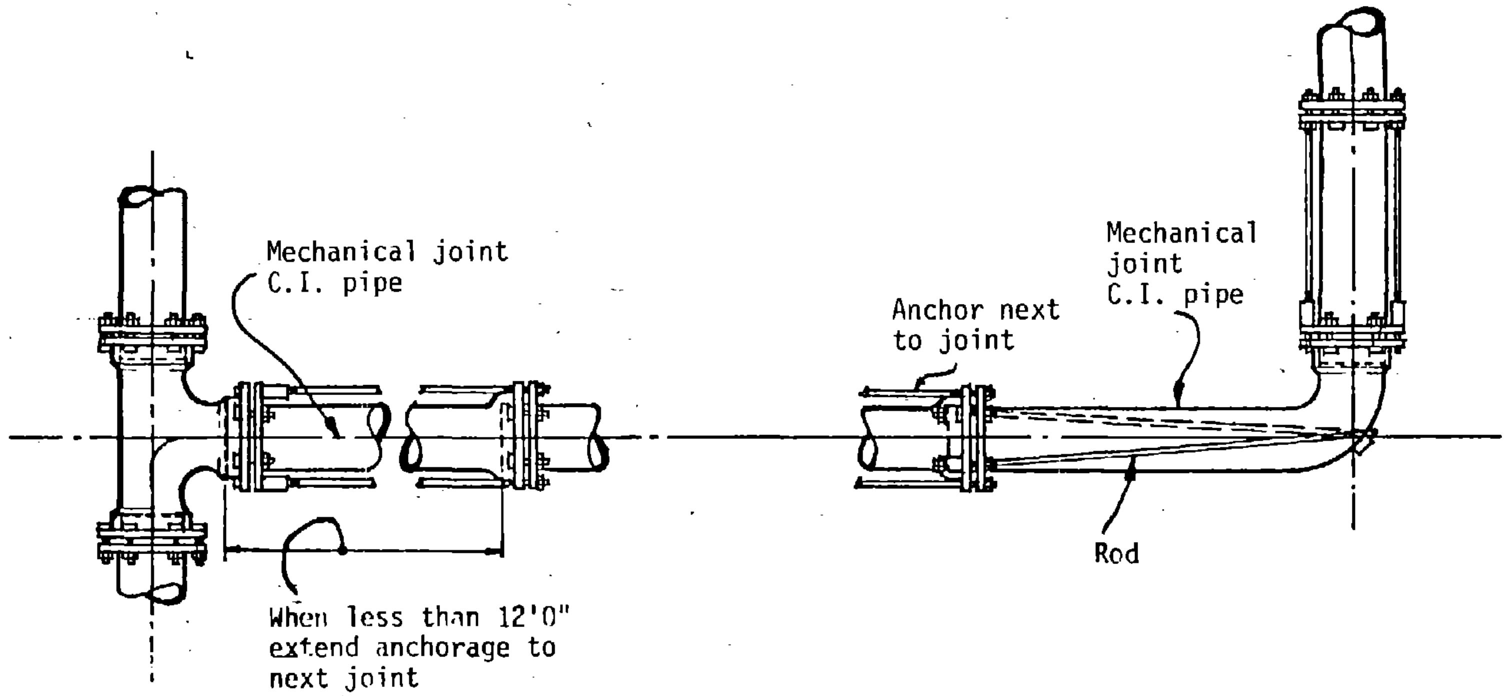


(a) Good practice

(b) Poor practice

Comment: Allow the pipe to pass through wall without restraint. Anticipate possible settlement of wall by providing sufficient clearance around pipe.

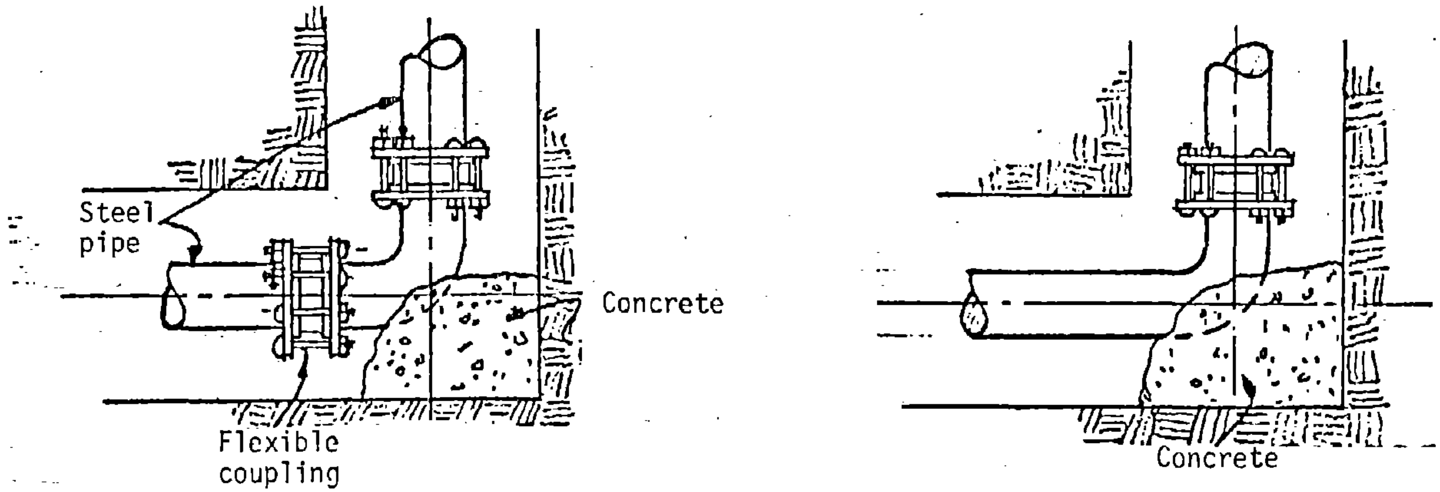
Figure VI-5. Retaining wall pipe penetration (157, 158).



(a) Tee (b) 90° bend

Comment: Shown above are two types of acceptable flexible joints. Since anchor blocks are not required, flexible connections are not necessary for all ends of the tee.

Figure VI-6. Flexible connection installations for tees and bends (157, 158).

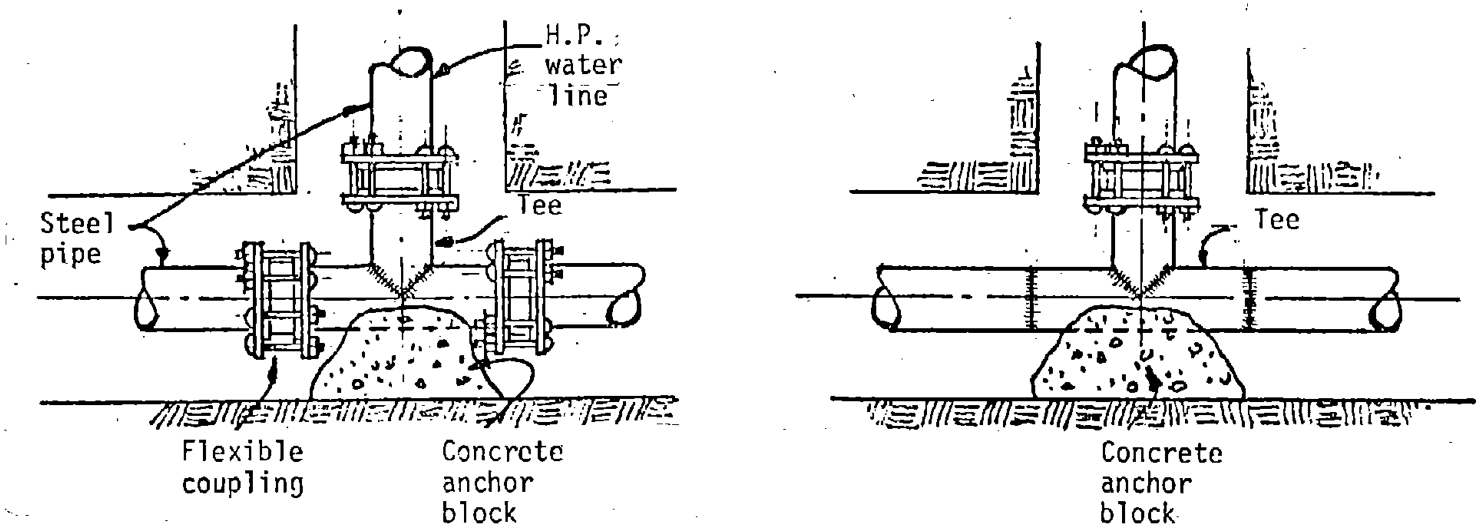


(a) Good practice

(b) Poor practice

Comment: For steel pipe, a flexible joint can be achieved by using a flexible coupling. Proper construction inspection, from a seismic standpoint, requires that concrete not interfere with the action of the flexible coupling.

Figure VI-7. Flexible coupling installation for bends with thrust block (157, 158).

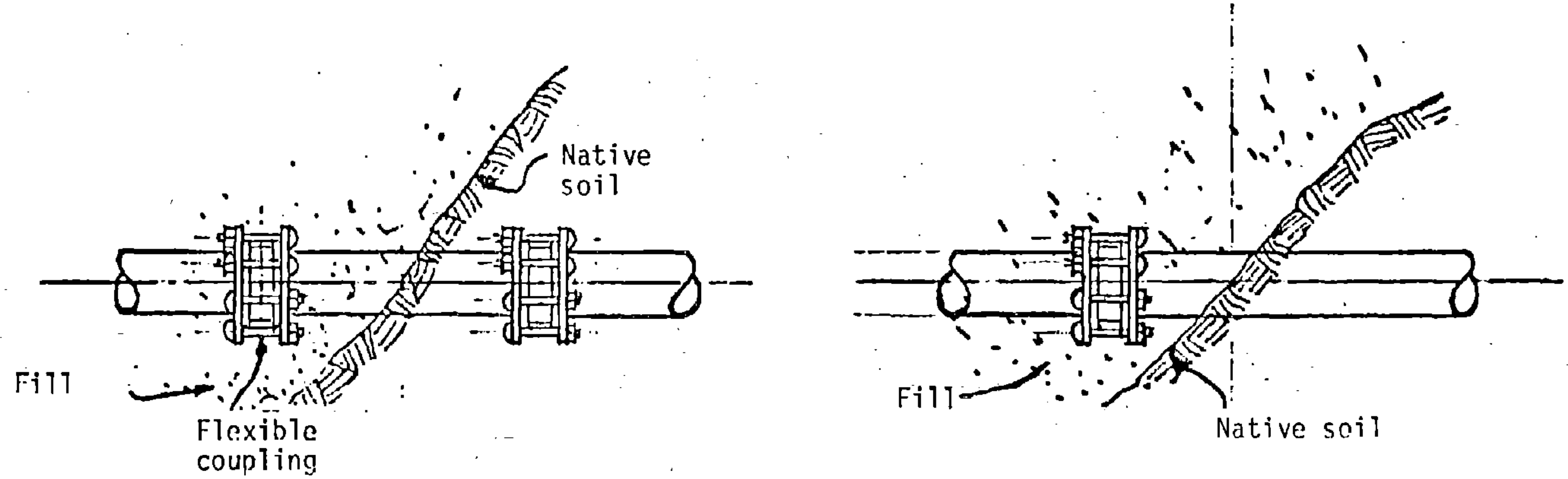


(a) Good practice

(b) Poor practice

Comment: Good seismic design practice requires the use of three flexible couplings at an anchored tee. The concrete anchor block used to prevent the high-pressure water line from separating also prevents movement unless flexibility is provided by flexible couplings.

Figure VI-8. Flexible coupling installation for tees with thrust block (157, 158).



(a) Good practice (b) Poor practice

Comment: Better flexibility is provided by the use of two flexible couplings, one on each side of the surface separating the fill and native soil.

Figure VI-9. Flexible coupling installation between non-homogeneous soil strata (157, 158).

Pipeline Corrosion

A pipeline weakened by corrosion is susceptible to damage in the form of leaks and larger blowouts when subjected to seismic shaking or ground deformations associated with ground failure and faulting. To protect buried metallic pipelines from corrosion, the following should be considered:

- coating or wrapping with coal tar or polyethylene to insulate the pipe from the surrounding soil
- lining ferrous pipes with cement or mortar or installing a plastic liner
- providing cathodic protection of the pipeline
- replacement of metallic pipe with non-corrodible pipe such as
  - asbestos-cement pipe (note: this pipe is very brittle and not suggested for earthquake prone areas)
  - plastic, polyethylene, or PVC pipe

Surge Pressures

Surge pressures (water hammer) may arise from two sources when considering seismic design of pipelines. An earthquake may cause a pump to stop and a check valve, which would otherwise be automatically controlled, to slam shut. The dynamic response of water in the pipeline may also increase pressure considerably.

Valves and fittings should be designed to withstand surge pressures, particularly where dead end piping occurs. Ductile iron castings have been recognized to be suitable to withstand significant surge pressures (044). Consideration should be given to installing pressure relief valves at critical locations.

River Crossings

River crossings, which sometimes involve siphons, are often constructed in areas of unconsolidated soils with high ground water levels. Such areas are very susceptible to liquefaction and earth slides, which impose loadings on these structures. To minimize the damage to such structures from earthquakes, the Japan Society of Civil Engineers has recommended the following design criteria (125):

- coated steel or ductile iron pipes should be used for river crossings
- flexible joints should be used for ductile iron pipe. When steel pipe is installed, flexible restrained joints should be used only at the bends of either side of the crossing
- pipelines approaching and following the crossing should have as gentle bends as possible, and concrete anchor blocks should be installed at the bends
- valves should be placed at both ends of the crossing

Similarly, the Japan Society of Civil Engineers has outlined specific design criteria for pipe bridges (125):

- the bridge-born pipe should be connected to the superstructure and equipped with expansion joints at each span
- on either side of the bridge where the pipe bends and attaches to a concrete abutment, the pipe should be fixed to the abutment with anchor bands and flexible restrained expansion joints should be installed at the connecting portions to the straight pipes before and behind the bridge (Figure VI- 10).

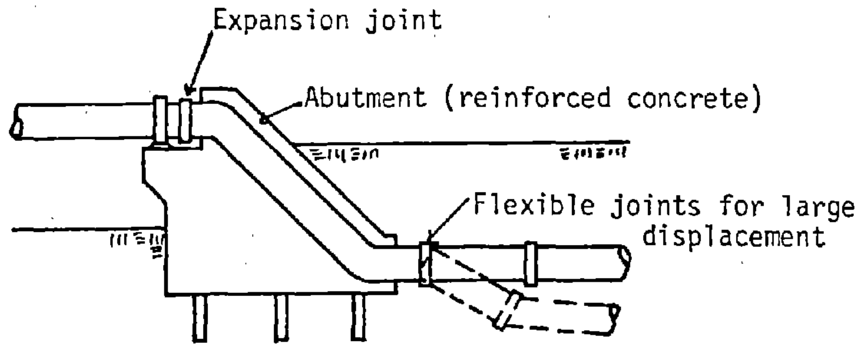


Figure VI-10. Attachment of pipes to abutments for pipe bridges (125)

- the foundation supporting the abutment should be reinforced with piles to reduce settling in soft soils
- valves should be installed on the pipeline before and behind the bridge

Service Connections

The use of plastic and copper pipe in place of galvanized and iron pipe for water services should enhance the capability to withstand seismic activity. Slack should be provided in the pipe when placing it in the trench to provide for movement.

Fault Crossings

Active fault crossings represent major hazards to pipelines. Fault displacements exert large compression, extension, and shearing movements on pipelines traversing the fault. A typical failure of a pipeline crossing a fault is shown in Figure VI-11 (044).

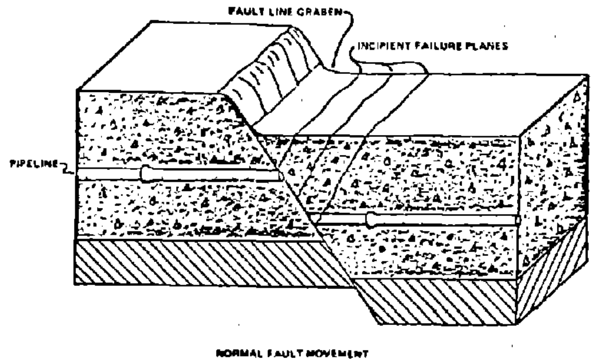


Figure VI-11. Failure of a pipeline due to fault displacement (044).

Damage to pipelines crossing active faults may be minimized by using ductile iron pipe joined with restrained expansion joints as described in the preceding subsection on flexibility of joints (See Figure VI- 2 - Restrained Expansion Connection). In addition to flexible joints, special backfill materials can be used; these have the capability to crush and deform in the event of earth shearing movements, but can support normal loads on the pipelines. These include plastic foams, foam concrete, and lightweight concretes composed of expanded vermiculite aggregates. An example of this type of design method for pipelines laid across faults is shown in Figure VI-12 (044).

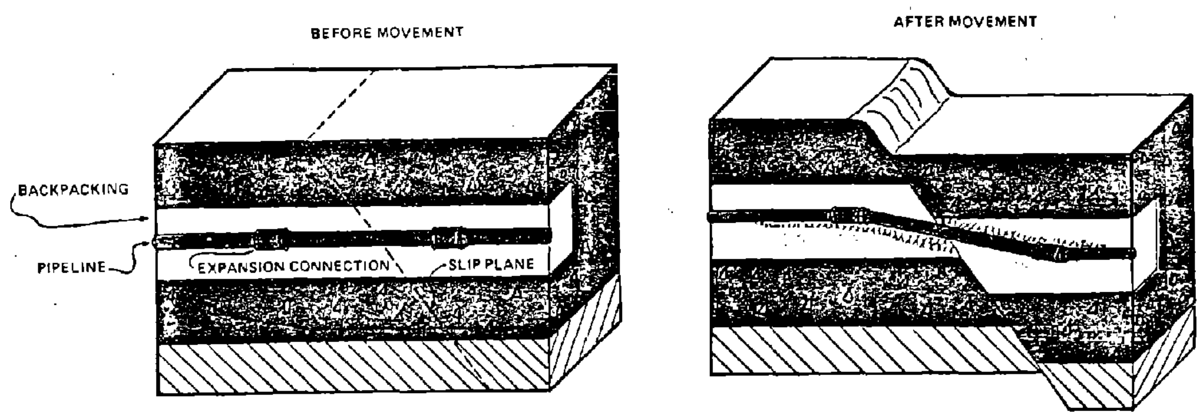


Figure VI- 12. Schematic diagram of pipeline design to withstand fault movement (044).

Other design considerations for pipelines crossing active faults include:

- add flexibility by installing ball joints in combination with restrained expansion joints (155).
- use ductile pipelines such as ductile iron pipe or welded steel pipe
- encase the pipeline in a tunnel to protect the pipe, as depicted in Figure VI-13.
- install a blow-off valve in the pipeline on the upstream side of the fault. Water can then be led to a reservoir for emergency storage after the blow-off valve fails.

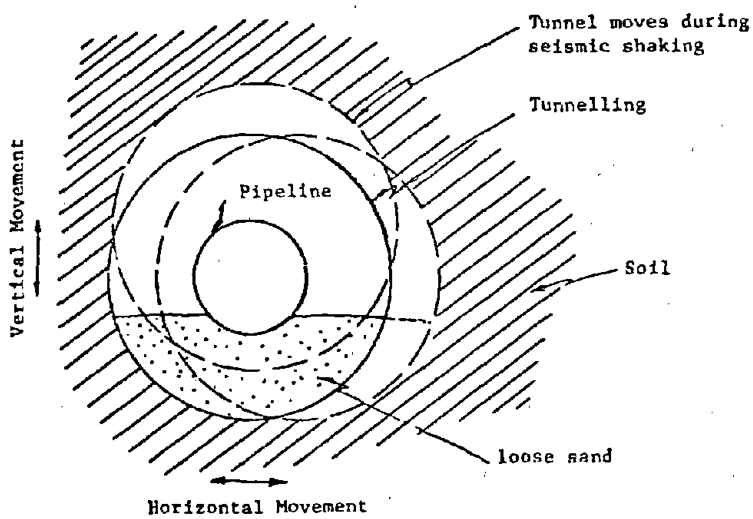


Figure VI-13 . Encasement of a pipeline in a tunnel to protect against damage caused by fault movement (155, 044).

Newmark and Hall (144) , studying pipelines crossing known faults, found that a pipeline that can be displaced out of a shallow trench with shallow sloping sides can withstand the transverse components of motion associated with a fault movement with only slight structural deformations (144). The shallow trench should extend for 300 feet on either side of the fault (if fault displacement equals 10 feet); only half that distance (150 feet) of trench is needed to withstand a 5-foot fault movement. Where fault movements are expected to more more severe (greater than 10 feet), the pipeline should be placed above ground with provision for freedom of movement between anchor supports (144). In general, a pipe's ability to withstand fault movements is greater when buried in soil than when buried in rock formations.

Generally, if faults with anticipated movements of 10 feet or less are crossed at right angles, the steel pipe of various grades can withstand these movements without failure, provided that the depth of cover is limited to 8 feet. If the fault crossing angle is unknown, depth of cover should be limited to 3 feet (144).

Kennedy (159) also studied the effects of fault movements on buried pipelines. His conclusions were essentially the same as those of Newmark and Hall; he made the following additional observations:

1) The further away the anchor points (points at which the pipe moves with surrounding soil, such as a bend in the pipe) of a buried pipeline are located on either side of the fault, the greater is the capability of the pipeline to withstand fault displacement (anchors or bends placed at least 200 feet on either side of the fault).
2) For constant pipe diameter, peak axial strain is nearly proportional to the inverse of the wall thickness. Thus, decreasing wall thickness considerably increases the axial strain. However, pipe diameter has a smaller effect on axial strain. A 40 percent increase in pipe diameter increases the peak axial strain by about only 10 percent.

DESIGN OF BOX CONDUITS AND OPEN CHANNELS

Box conduits and open channels (large transmission structures) have been heavily damaged in past earthquakes by ground deformations associated with faulting and ground failures. There are basically two general design considerations for such structures if subject to major ground displacements:

- design the structure with as much flexibility as possible in order to withstand imposed ground deformations
- and/or design the structure to fail at a predetermined point and allow for oversizing to facilitate repair of the structure

Calculation of the earthquake induced forces on buried tank walls have been included in Chapter VII, Section D. Similarly, design considerations for tank walls to withstand earthquake induced loadings are given in Section C of this chapter.

One of the most common failures of box conduits is cracking of the walls due to non-alignment of construction joints. If construction joints are aligned (i.e., in the same vertical plane at invert, soffit and walls), a clean break will occur. In addition, longitudinal cracking of walls can be reduced by more frequent spacing of construction joints to allow the conduit, by translating and rotating as a rigid body, to accommodate fault motion, settlement of surrounding soil or slippage due to slides. The following design recommendations for box conduits were developed by the U.S. Army Engineer District of Los Angeles following the 1971 San Fernando earthquake (046).

- the construction joints of invert, walls, and soffit should be in the same vertical plane, and spacing should be 30 feet maximum
- shearing resistance of the joint should be such that $2R \leq gL$, where R is the shearing resistance, g the least transverse design load per linear foot of box, and L is the distance in feet between joints.

- seismic joints (such as those depicted in Figures VI-18) can be used to allow for longitudinal shortening. The joints divide the box into segments by providing a continuous weak band all around the box while maintaining the minimum strength required.
- waterstops should be used in construction and expansion joints to protect against loss of the water tight integrity of the joint due to separation
- if the conduit is located in area susceptible to liquefaction, the potential increase of lateral loads on the conduit walls should be included in design
- oversizing the conduit located in a known fault zone can protect against loss of discharge capacity caused by a reduction in cross sectional area and enable repairs to be completed more readily.

Open channels are similarly affected by seismic earth movements. In general, all of the design considerations for box conduits apply to open channels.

C. TREATMENT PLANTS, PUMP STATIONS, AND EMERGENCY POWER SUPPLIES

Treatment facilities are the most complex parts of water and sewage systems from an equipment standpoint. This section discusses design considerations for buried tanks and vaults, equipment, plant piping and building structures. Pump stations and emergency power supplies are also included because of the similarities in the types of structures and equipment involved.

Geotechnical aspects of tank design are discussed in this section, including soil stabilization techniques. General concepts are presented on seismic resistant tank wall design, followed by a subsection dealing with detailed design. Seismic resistant design of tank appurtenances is also discussed.

Equipment is discussed both from a general standpoint and according to equipment type. The various parameters affecting equipment response to earthquake motions are listed, and 13 categories of equipment types are discussed in relation to possible seismic resistant design approaches. Equipment anchorage and connections to other systems are included. Equipment qualification and testing, significant aspects of seismic resistant technology in the nuclear industry, are also discussed.

The response of suspended piping to seismic motions is discussed in detail. A wide range of flexible pipe joints and wall penetrations are considered in accordance with their applicability to specific installations.

A short discussion on building design is included, but only as it effects the operation of the systems they house. Detailed building design is beyond the scope of this report.

BURIED TANKAGE AND VAULTS

This section includes design considerations for concrete water and sewage treatment facility structures extending below the ground surface. The major types of structures considered are listed in Table VI-1.

Hydraulic structures (containing water) and vaults (dry structures) can be designed on the basis of similar considerations, although forces induced on a tank from the water inside would not be considered in the design of a dry chamber.

Buried tank seismic design should consider tank settlement, earth retaining forces, tank flotation, effect of the inertia and sloshing of the water, inertia of the tank structure and the soil it supports, and appurtenant items. Sloshing and the inertial effect are earthquake induced forces only; the other items, although normally considered in a non-seismic design, must now be considered in greater detail.

Geotechnical Considerations

Siting -

When a new plant site is proposed, a geotechnical study locating fault traces and areas of potential soil liquefaction, densification and other geologic hazards should be performed. Although plant siting was considered in Chapter V, it is particularly significant in the design of buried concrete and steel tanks, as these massive structures require stable bearing strata for support. If the site is vulnerable to soil densification or liquefaction, three courses of action are available: (1) an alternate site can be used if a more favorable one can be located; (2) the high densification and liquefaction vulnerability can be ignored if the probability of an earthquake occurring during the design period multiplied by the potential cost

TABLE VI- 1. MAJOR WATER AND SEWER SYSTEMS HYDRAULIC STRUCTURES AND VAULTS

Water Systems-Hydraulic Structures

Screen Chambers	Disinfection Contact Basins
Rapid Mix Tanks	Filter Basins
Flocculation Tanks	Clear Wells
Clarifiers (Settling Tanks)	Below Ground Reservoirs
Conduits and Channels	

Water Systems Vaults

Pipe Galleries	Building Foundations & Basements
Pump Station (Below Ground)	Meter Pits

Sewage Systems-Hydraulic Structures

Equalization Basins	Reactor Basins (Biological/ Chemical)
Clarifiers (Primary, Secondary, Advanced)	Sludge Holding Tanks
Sludge Digesters	Conduits (Open Channel Metering)
Sludge Thickeners	Disinfection Contact Basins
Pump Station Wet Wells	Grit Chambers
Filter Basins	Comminutor Basins

Sewage Systems Vaults

Pump Stations (Dry Well)	Meter Pits
Pipe Galleries	Building Foundations & Basements

of damage is less than construction to resist seismic induced conditions; and (3) the facility may be designed to resist the effects of densification or liquefaction or the vulnerability may be reduced with corrective measures. Past studies have shown the cost of soil stabilization to be very high for a complete facility; therefore, proper siting can be critical in minimizing seismic damage.

Alluvial plains are susceptible to liquefaction because of the soil type and high water table. Sewage treatment plants are typically sited in low areas, often on alluvial plains, to permit gravity flow through sewers. Water treatment plants, which are sometimes located adjacent to major water bodies to provide access to raw water, may also be located on alluvial plains.

Obviously, siting of water or sewage treatment plants in areas of high liquefaction or densification potential can be a serious problem. If at all possible, no structure or system component should be placed on earth fill or on an existing stratum with a high potential of liquefaction or densification, or near the toe or shoulder of a slope susceptible to failure by earthquake effects (125).

Settlement -

Settlement of structures can occur when the supporting stratum is densified by shaking, or when liquefaction occurs. A geotechnical study should be performed on all proposed facility locations to determine the soil characteristics and the densification and liquefaction potentials. (Soil stability is discussed in Chapter V, System Siting, Planning and Conceptual Design.) If the decision has been made to construct the facility over a stratum with a reasonable liquefaction or densification potential, the engineer may alter the soil characteristics or may design the structure to overcome earthquake effects.

Densification and liquefaction induced by vibration occur primarily in uniformly graded cohesionless soils with low relative density. Liquefaction may begin when densification occurs in the soil mass below the ground water table. It should be noted that the effects of densification are less likely to be catastrophic than the effects of liquefaction. Densification causes the soil to compact, but the bearing capacity of the strata is maintained. When liquefaction occurs, the soil mass becomes a liquid, allowing lateral flow of the soil mass.

Every attempt should be made to design a facility to minimize settlement from earthquake induced motions. Because it is difficult to eliminate settling entirely, continuity should be maintained across the supporting strata so that the relative settlement of component parts can be minimized, thus limiting damage.

Compaction -

A soil must be relatively loose or uncompacted for settling to take place. This condition is typically found in fill areas and alluvial deposits. A soil's relative density, D_r, can be measured in terms of its void ratio, e, where:

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}}$$

where: void ratio is the ratio of the volume of the space between the soil particles to the volume of the soil particles

e_{max} = void ratio of the soil with minimum density

e = in situ void ratio

e_{min} = void ratio of the soil with maximum density

Soils with a D_r of less than 50 percent may be highly vulnerable to failure, depending on other conditions. Soils with a D_r above 70 percent are quite stable (109). For a more detailed analysis of the conditions required for soil failure, refer to Chapter V.

Increasing the relative density of a soil to above 70 percent greatly reduces the liquefaction potential. This can be accomplished by removing the soil and replacing it with adequately compacted soil, or by compacting the soil in place.

The least expensive method for compacting deep non-cohesive soil appears to be the use of vibroflotation (simultaneous deep vibration and saturation of the soil). This method is limited to soils containing less than 20 percent silt and clay, but can achieve a D_r of 70 percent if applied correctly (109). Vibroflotation equipment consists of a cylinder 2 to 3 meters long, 25 to 40 centimeters in diameter and weighing 2 to 3 tons, equipped with water jetting nozzles on both ends and a motor for inducing vibration. The cylinder is jetted with water into position while vibrating to the bottom of the soil layer requiring consolidation, the cylinder being oriented vertically. Once in position, vibration at a speed of 1800 cycles per minute consolidates the soil in a 1 to 2 meter radius. Sand is fed from the top as the soil is consolidated, while the cylinder is slowly raised at a rate of about 10 minutes per meter. A pattern of application of 6 feet to 8 feet on center is usually recommended (114, 109, 160). Figure VI-14 shows a range of soil grain sizes suitable for compaction by vibroflotation.

Other forms of vibration such as blasting may be used to consolidate soil.

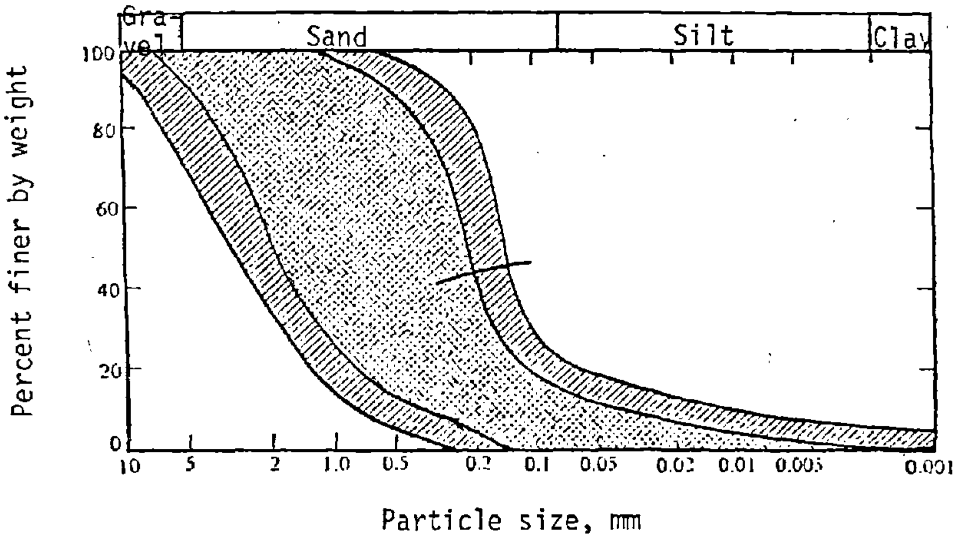


Figure VI-14. Range of soil grain sizes suitable for compaction by vibroflotation (109).

Drainage -

For liquefaction to occur, the stratum vulnerable to the initiation of liquefaction must be below the ground water table. Depending on the local topography, an effective drainage system may be constructed. Following the 1971 San Fernando earthquake, vertical gravel columns were constructed at the Joseph Jensen Water Filtration Plant to alleviate pore water pressure that could develop in the strata during an earthquake.

Grouting -

For significant settlement caused by earthquake vibration to occur, the soil must be non-cohesive. The cohesiveness of the soil may be increased by grouting between soil particles through pressure grouting or intrusion grouting with cement, chemicals or bitumen. Soil stabilization techniques are dependent on the specific soil characteristics. The size of the soil particles, the moisture content and the chemistry of the soil are all important parameters. An analysis of the effectiveness of soil stabilization is necessary because some soil conditions are not adaptable to any of the usual techniques. Chemical grouting is usually used in partially saturated soils. There are, however, some instances in which chemical grouting has been successfully used in dry, granular or fractured soils (161). Cement grouting is most successful in gravelly sand with particle sizes greater than 1.5 mm. Bitumen grouting is used primarily to seal the soil stratum (114).

Sensitive clays -

Sensitive clays are clays that are vulnerable to instantaneous structural failure from increased loading due to a breakdown of the bond between the soil particle and the bonded water. The effect of earthquake induced motions on sensitive clays should be included in the site geotechnical study. (A detailed description of sensitive clays is included in Chapter V.) Sensitive clays may be removed and replaced with a stable fill material. Structures may be supported on piles extending through the sensitive clay layer. The permeability of sensitive clay is too low to permit successful grouting. Attempted consolidation by vibration or preloading would cause the clay to "liquefy" but not to stabilize adequately. Electrochemical stabilization has been used for clays using aluminum anodes which migrate into the soil (114). A soil stabilization specialist should be consulted for a detailed analysis of the situation if necessary.

Piling -

The alternative method of mitigating earthquake induced settlement is to attain tank bearing on a stable soil layer below the layer vulnerable to consolidation and liquefaction. The load can be transferred from the bottom of the tank to the stable soil layer by end bearing piles. Orange County

Sanitation District's Water Factory 21, for example, is built on a series of 780 piles because of the high liquefaction potential of the local sandy soil.

Ductility should be provided in piles for seismic induced bending stresses (162).

Japanese codes require that the tops of piles be inbedded into the structure as deeply as possible in order to transfer all possible forces (125).

The requirement that "individual pile caps and caissons of every building or structure shall be interconnected by ties, each of which can carry in tension and compression a minimum horizontal force equal to 10 percent of the larger pile cap or caisson loading, unless it can be demonstrated that equivalent restraint can be provided by other approved methods" is included in the Uniform Building Code (140) with a similar requirement in the Applied Technology Council's Tentative Provisions for the Development of Seismic Regulations for Buildings (ATC-3) (141).

Extreme caution should be used in the design of a structure that is partially supported by piling and partially supported by an alternate method. Continuity of the structure's response from earthquake motions can best be attained by using similar foundation supports throughout the structure. If a structure is supported on two unlike foundations, a flexible joint between the two sections may accommodate any differential settlement between the two structures (125).

Wood piles should be avoided where they extend above the minimum ground water table to avoid foundation deterioration (125).

Tank Wall Design Considerations

Backfill -

Calculation of earthquake induced forces on tank walls have been included in Chapter VII, Section D. The engineer can control the type of backfill used behind the tank walls during construction. A non-cohesive soil is normally used, as it is easier to attain a specified compaction with a minimum effort. The relative density of the backfill has a significant effect on the liquefaction potential of the backfill material. The most severe condition that must be resisted by a retaining wall is liquefaction of the backfill, essentially containing a very dense fluid. It should be noted that the inertial effect of the liquefied soil is disregarded, as liquefaction is not expected to occur until after the shaking has stopped. The liquefaction potential can be controlled by using a backfill that is compacted to a high relative density. This can be achieved by using a well-graded backfill. Detailed soil compactibility information can be found in most foundation engineering texts.

Lateral forces -

The inertial forces of the tank structure and the soil supported by the tank structure must be taken into account. The fill over a tank structure, possibly being used to prevent flotation, may exert a substantial lateral force induced by earthquake motions.

Flotation -

Standard non-seismic design procedures include provisions to resist flotation from the maximum ground water table when the tank is empty. However, earthquakes' motions causing liquefaction of backfill and virgin soil immediately adjoining the tank structure may cause the tank to float, even if it has been designed to resist flotation from high ground water. The net buoyant force of the tank can be calculated as the tank displacement (if empty) multiplied by the unit weight of the surrounding liquefied soil mass, less the weight of the tank, soil overburden, and supported equipment. Once liquefaction has occurred, the conditions for flotation are far more favorable than under normal conditions. The specific gravity of the liquefied mass is much greater than that of water. The effect of a lip on the tank foundation keying the tank into the surrounding soil mass is lost.

The buoyant forces causing flotation can be controlled in several ways. They can be resisted by providing a positive tie down mechanism, achieved by tying the tank to supporting piles designed to resist uplift. Care must be taken to provide for the transfer of the buoyant force to the top of the piles.

The buoyant forces can be balanced by increasing the weight of the structure by using mass concrete, heavy aggregate concrete or adding overburden above the tank. This method of resisting flotation from liquefaction is not recommended because it is difficult to equalize the buoyant force which may lead to differential settlement. This method may be useful in conjunction with piles providing bearing but not resisting uplift.

Detailed Design of Tanks

Detailed design considerations applicable to concrete tank and vault design have been collected from a number of seismic and concrete design codes. Where design details from other sources were not available for a particular application, the authors have proposed design details using materials and methods used in other similar situations. The following major sources were used:

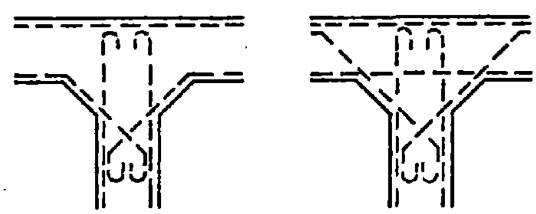
A. Japan Water Works Association, The 1st Subcommittee on Countermeasure for Earthquake Disasters (JWWA) (125)

B. American Concrete Institute, "Concrete Sanitary Engineering Structures," Committee #350 report (ACI-350) (163)

C. East Bay Municipal Utility District, "Engineering Standard Practice, Seismic Design Requirements," ESP #550-1 (EBMUD) (162)

Layout and concrete design -

1. The simpler the design of a structure, avoiding abrupt changes in configuration, the more resistant it is to earthquake motions. For example, a circular tank is preferable to a rectangular one (JWWA).
2. Sharp interior angles should be avoided in concrete tanks (JWWA).
3. The floor and wall of a basin should be constructed as a unit (JWWA).
4. A reinforced concrete rigid frame structure shall not have the arrangement shown in Figure 15a below, which may allow pullout of the corner bars; that shown in Figure 15b would be more appropriate (JWWA).



(a) Poor practice (b) Good practice

Figure VI-15. Rigid frame joint reinforcing (125).

5. Concrete walls retaining water or sewage should be designed on the basis of elastic distribution of stresses.

6. Tank design shall consider the provisions of "Concrete Sanitary Engineering Structures" reported by American Concrete Institute Committee. (ACI-350).

7. Reinforced concrete moment frame design resisting major seismic induced forces should inhibit:

   a. Shear failure

   b. Compressive failure without concrete confinement (spiral or hooped transverse reinforcing)

   c. Joint shear failure, i.e., at beam-column joints

   In general, the use of moment frame systems may not be appropriate due to their flexibility, but if they must be used, adherence to these principals will materially improve the structural performance of the frame during major earthquake shaking.

8. The allowable concrete and steel working stresses are shown in Tables VI-2 and VI-3 , respectively. These allowable stresses are included in the ACI Committee 350 report for "Concrete Sanitary Engineering Structures", conservatively developed to prevent cracking of the structure. "All concrete shall have a 28 day compressive strength of 3500 psi where the concrete is not exposed to severe and frequent freezing and thawing and 4000 psi where the concrete is exposed to severe and frequent freezing and thawing, except where special structural or other considerations require concrete of greater strength " (163).

TABLE VI-2 - RECOMMENDED WORKING STRESSES FOR ALL STRENGTHS OF CONCRETE IN SANITARY ENGINEERING STRUCTURES WHICH MUST BE WATERTIGHT AND RESISTANT TO CHEMICALS (163)

Description	Recommended value
Modulus of elasticity ratio; n	9
Flexure; f_c	
Extreme fiber stress in compression	1350 psi
Extreme fiber stress in tension in plain concrete footings and walls	88 psi
Shear; v (as a measure of diagonal tension at d from face of support)	
Beams with no web reinforcement	60 psi
Joists with no web reinforcement	66 psi
Members with web reinforcement or properly combined bent bars and vertical stirrups	274 psi
Slabs and footings (peripheral shear)	110 psi
Bearing:	
On full area	750 psi
On one-third area or less	1125 psi

9. Reinforced concrete walls 10 feet or higher which are in contact with liquid shall be at least 12" thick (ACI-350).
10. A larger number of smaller reinforcing bars is generally more favorable than an equal area of larger bars (except where congestion occurs), with the maximum bar diameter being less than or equal to 6 percent of the wall thickness (ACI-350).
11. Minimum reinforcement steel in walls should be as specified in ACI-318-77 (164). (ACI-350).
12. Moment resisting columns, or columns cast integrally with the floor and roof members, shall have spiral reinforcement in accordance with the current edition of the Uniform Building Code (EBMUD).

TABLE VI-3. RECOMMENDED STRESSES AT SERVICE LOAD FOR RECOMMENDED MAXIMUM 12-IN. SPACING OF REINFORCING BARS IN SANITARY STRUCTURES (163)

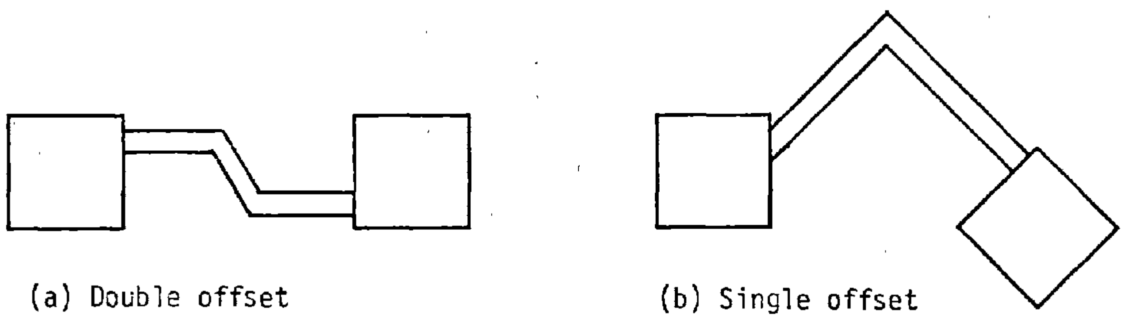
Bar sizes	Sanitary structure exposure condition* and maximum Z-value+	Maximum stress at service load, psi
All sizes	Members in direct tension	14,000
#3, #4, #5	Flexural members. Very severe exposure. (Maximum Z = 95)	22,000
	Flexural members. Normal sanitary exposure. (Maximum Z = 115)	27,000
#6, #7, #8	Flexural members. Very severe exposure. (Maximum Z = 95)	18,000
	Flexural members. Normal sanitary exposure. (Maximum Z = 115)	22,000
#9, #10, #11	Flexural members. Very severe exposure. (Maximum Z = 95)	17,000
	Flexural members. Normal sanitary exposure. (Maximum Z = 115)	21,000

* Very severe exposure is defined as the face exposed to liquid sewage retention or condensation of fumes from sewage, plus freezing-and-thawing and wetting-and-drying. Normal sanitary exposure is defined as the face remote from corrosive conditions above or exposure in water storage or water treatment plants.

+ The Z-values referred to are defined in ACI 318-77 (164). Derivation for crack control formulas are in the Commentary to ACI 318-77 (164).

Concrete wall joints and pipe penetrations-

1. Tanks, structures, or parts thereof employing great differences in rigidities or geometries shall be separated by a flexible joint (EMBUD).
2. Tanks, structures, or parts thereof founded in different foundations shall be separated by a flexible joint (EMBUD).
3. When walls are designed as shear walls or diaphragms, expansion and construction joints shall be keyed to carry the required shear forces (EMBUD).
4. On structure side walls, joints shall be placed near the corners (JWWA).
5. Some flexibility between two tanks connected by a concrete channel or pipe may be attained by providing an offset as shown in Figure 16.



(a) Double offset

(b) Single offset

Figure VI-16. Channel or pipe offsets.

6. Concrete wall flexible joints to accommodate concrete shrinkage and thermal expansion and contraction shall not be mroe than 60 feet apart for members exposed to the atmosphere and 100 feet for members completely underground. Recommended minimum joint widths not taking into account seismic motions are shown in Table VI-4.
7. To maintain water-tight integrity following an earthquake, flexible joints should include flexible waterstops. Flexibility of the joint will be dependent on the distance between the concrete and the flexibility of the waterstop. Flexible P.V.C. waterstops are commercially available

TABLE VI-4 - RECOMMENDED EXPANSION WIDTHS, INCHES (163).

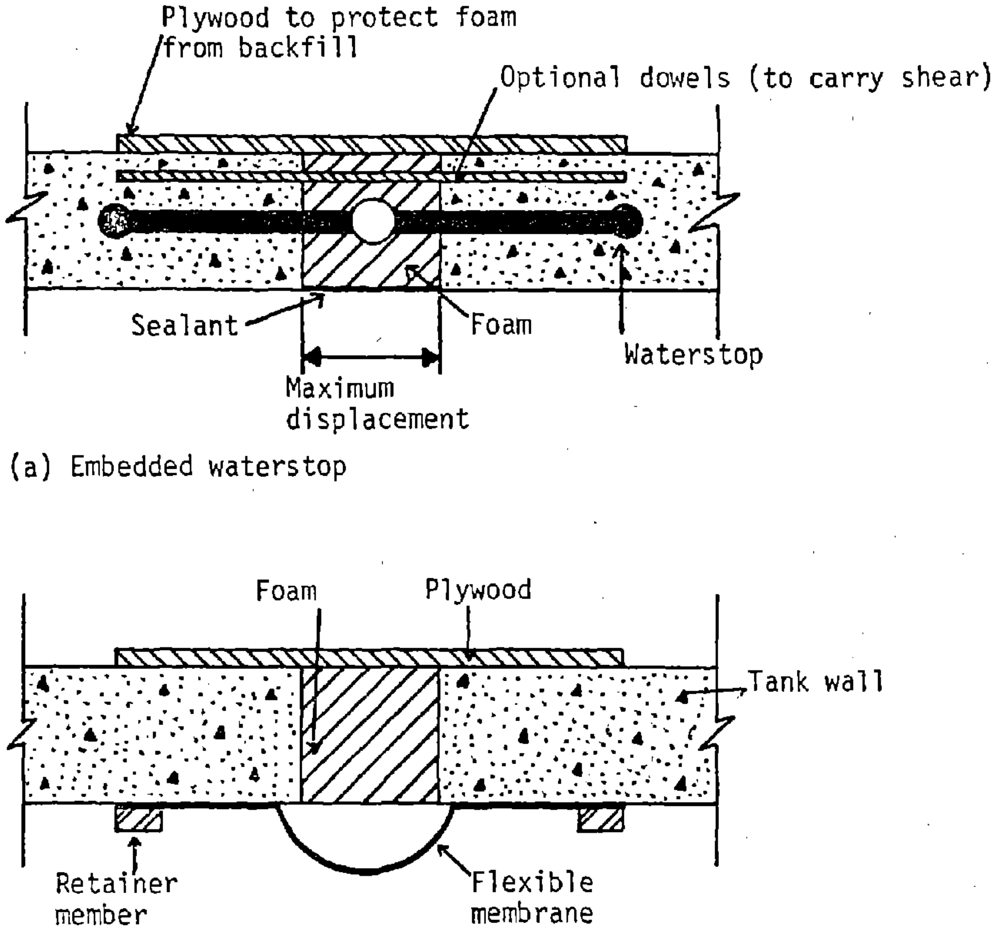
Temperature range	Spacing between expansion joints			
	40 ft	60 ft	80 ft	100 ft
Underground, 40 F	1/2	3/4	7/8	1
Partly protected, above ground, 80 F	3/4	7/8	1	*
Unprotected, exposed roof slabs, etc, 120 F	7/8	1	*	*

*Not recommended (ACI-350)

as wide as 9 inches with mechanical elongation as much as 1½ inches (165). Wider stops with greater elongation capabilities could undoubtedly be manufactured if required by the water and sewer industry.

8. Joint flexibility can be limited to the direction required, carrying loads on other directions by making use of dowel rods, etc.
9. Flexible joints are included in the design to compensate for earthquake induced ground strains transferred to the structure and the structure's resonant vibratory motions relative to one another. Flexibility required to accommodate ground strains can be calculated in accordance with the equations presented in Chapter IV for pipeline strains. The amplitude of the structure's resonant vibrations will depend on both the structure and surrounding soil's rigidity.
10. Flexible joints should be spaced close enough together to limit tank wall material stresses to less than their design stress. A maximum joint spacing of 30 feet has been suggested for channels (046).
11. Two possible flexible joints for tanks are shown in Figure VI-17 a and b.

The joint shown in Figure VI-17a makes use of a standard PVC waterstop.



(a) Embedded waterstop

(b) Diaper

Figure VI-17. Flexible joints.

Deformable foam is applied to the void to prevent buildup of solids. The foam should be sealed on the water side and protected from backfilling on the outside with plywood or plasterboard. The joint will be flexible in all three orthogonal planes. Flexibility of the joint can be limited to one plane by inserting dowel rods similar to those used in constructing concrete roadway joints. By making use of dowels, the joint is capable of carrying shear.

The joint shown in Figure VI-17b can be installed after a tank has been poured. The "diaper" seal would be made of a heavy PVC liner material attached to the tank walls with flat bars anchored to the wall, pressing the diaper against the wall. A sealant should be used between the wall and diaper. Foam, plywood, and a foam sealant should be used with the use of dowel rods being optional as with the joint shown in Figure VI-17a. The "diaper" joint could be applied to a tank retrofit.

12. A possible rigid seismic joint is shown in Figure VI-18. This joint would be used when adequate flexibility in compression cannot be attained by using a flexible joint. This joint must be used in combination with a flexible joint to allow for thermal expansion and contraction and concrete shrinkage. The joint is designed to fail before the tank or channel wall, limiting repair to a small area. The joint should be used in the same vertical plane across the structure. The shearing resistance of the joint should be limited to less than one-half the minimum transverse design load to assure joint failure before the wall (166, 046).

13. Pipe penetrations through walls shall be flexible. The penetration design shall allow longitudinal movement as well as small angular movement. The design in Figure VI-19a may be used against a hydraulic head whereas the design in Figure VI-19b is primarily to prevent the entrance of moisture and occasional water under low hydraulic heads. Figure VI-19a's design may be installed under wet conditions and may be tightened if leakage develops. That shown in Figure VI-19b is designed to fail before the wall or pipe if significant movement takes place, at which time it must be replaced. A flexible sleeve or joint cast into the concrete, commonly used for manhole pipe penetrations, can be employed. A detail

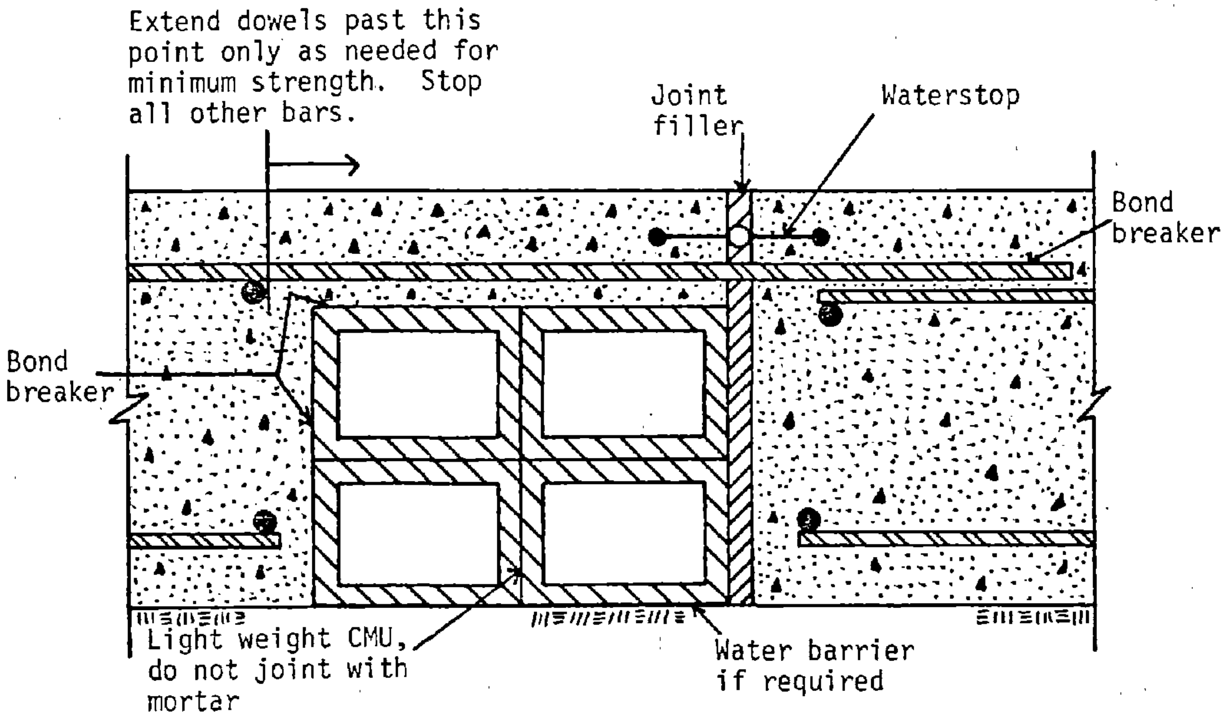


Figure VI-18. Rigid seismic joint (046).

is shown in Figure VI-3. For detailed piping designs, refer to the "Plant Piping" subsection presented later in this section.

14. Collection troughs (launders) found in clarifiers and filters are subject to impulsive and convective forces from the surrounding water during an earthquake. These forces can be calculated by the procedure presented in Chapter VII. The joints connecting these troughs to tank wall penetrations or other troughs should be flexible. They should be designed so that they will not damage the tank wall if they fail and so that they may be easily replaced.

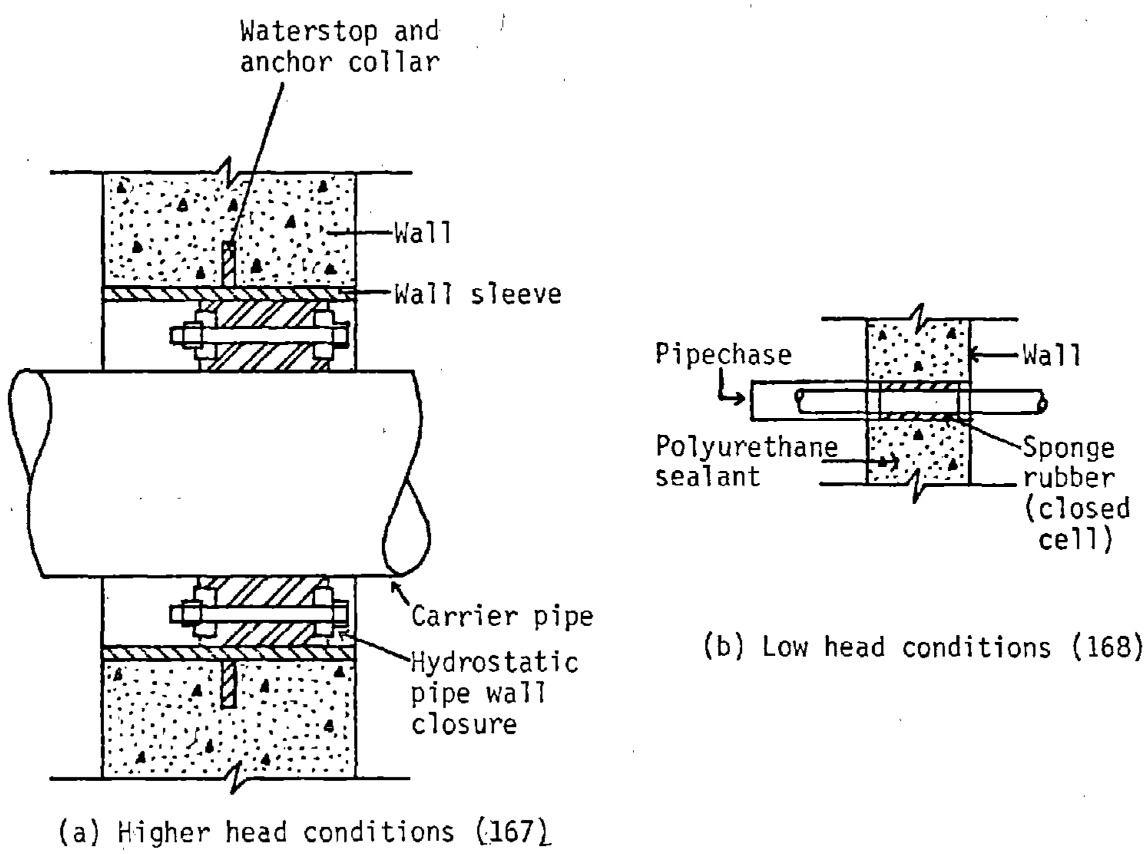


Figure VI-19. Flexible pipe wall penetrations.

Tank Appurtenances

Lateral forces from water -

Internal components of tanks are subject to impulsive and convective forces from the surrounding water as well as the inertial force from their own mass during an earthquake. Major components include:

Collection troughs (launders)	Flocculator shafts and paddles
Baffles	Mixer impellers
Center Wells	Aerators
Piping	Clarifier tubes or plates
Sludge scrapers and drive mechanisms	Weirs

A procedure for calculating these forces is presented in Chapter VII, Section D of this report.

The engineer has two alternatives in considering these forces: (1) design to resist; or (2) allow to fail and provide for rapid repair. For some components such as piping, aerators, and shafts and impellers, the increased cost of making the structure more resistant to these forces may be minimal and should be included in the design. Components such as baffles and collection troughs have a large cross section and are relatively weak horizontally. Strengthening these components to resist these horizontal forces may be costly. Depending on the estimated earthquake recurrence during the facility's design period, the designer may opt to allow these components to fail during an earthquake. If this is the case, the design should attempt to limit damage to the tank structure from component failure. Provision should be made to allow for quick replacement of the components if they are damaged. Availability of the components from the supplier or ease in local fabrication should be kept in mind. In deciding whether to permit failure or resist the forces, the critical nature of the equipment and the operating goals following the earthquake should be considered.

Other forces -

Platform mounted surface aerators may encounter a special problem. Tank sloshing creates an unlevel liquid surface. If a steep hydraulic gradiant across the impeller is encountered, the pumping energy transferred to the water through the impeller may become asymmetric, causing irregular forces on the aerator driving equipment.

Tank bridges -

Tank bridges are commonly found on clarifiers, flocculators and mixers to support the drive mechanism and piping for sludge withdrawal, support the mixer and flocculator units, and provide access to the center of the tank for monitoring and maintenance.

Acceleration forces induced in bridges and the supported equipment should be calculated in accordance with Chapter VII, "Determination of Seismic Induced Loadings." In addition to these forces, the impulsive and convective forces transferred from the immersed portion of the equipment should be included.

Shaking of the tracks supporting travelling bridges may cause wheels to leave the track, making the system inoperative. Consideration should be given to provisions to positively hold the wheels on the track.

GENERAL EQUIPMENT DESIGN CONSIDERATIONS

The purpose of this subsection is to consider general approaches for mitigating earthquake induced damage to equipment. Earthquake induced loadings can be calculated using the procedures presented in Chapter VII. This subsection presents methods to reduce these induced loadings and to resist their effects. General considerations include equipment layout, support, geometry, response, rigid and vibration isolation type anchorages, interconnections, backup systems, and qualification and testing.

While the design considerations included herein are most critical for pieces of equipment required to maintain the facility's operation during and following an earthquake, their application to all equipment may mitigate damage to non-essential equipment as well. The designer should keep in mind that the failure of a structure adjacent to a critical one may disrupt the operations of the critical component; if so, both equipment elements should be considered critical.

In most cases, equipment design and anchorage is left up to the equipment manufacturer and contractor, with only superficial review by the design engineer. However, detailing of equipment and anchorage is critical for seismic resistant design and should therefore be considered in that light by the system design engineer.

Deflection or drift of a structure is of concern, particularly for a flexible structure where substantial deformation may occur. Such a structure may interact with adjacent equipment, causing damage. An example of this situation is a storage tank or bin extending through the floor of a room above; clearance should be allowed for in the floor penetration. Attachment of structures to both the floor and ceiling should be carefully designed, as differential displacement may also take place between the structure and the ceiling due to their respective different response characteristics.

Massive equipment should be located on the ground floor or basement of a structure if possible. The acceleration to which a mass is subjected may be amplified by its supporting structure, i.e., the building structure. Provision for amplification has been included in calculation of the seismic induced loading in Chapter VII. However, it is recommended that this situation be avoided if possible. If the mass of the equipment is large enough and located above the ground floor, it must be added to the building mass to determine the seismic response of the building. Massive sludge dewatering equipment such as filter belt presses and centrifuges are often mounted on the second floor of a structure to allow for loading of sludge into trucks by gravity. Location on the ground level could be accommodated by using a belt conveyor for loading trucks from sludge dewatering equipment. Emergency power generators, massive pieces of equipment sometimes found on second stories, should also be mounted on the ground floor or in basements.

Special care should be taken in locating equipment, storage tanks, and feed lines containing hazardous materials. Most regulatory agencies require that chlorination facilities be located in a separate, well-ventilated room away from the rest of the treatment facility. Similar precautions should be taken with other types of systems carrying caustic, acidic or otherwise hazardous materials. The most stringent provisions should be used to avoid earthquake damage to these facilities. However, it is not possible to entirely eliminate all earthquake hazards (i.e., to make a facility "earthquake proof") due to the highly variable nature of earthquake phenomena. Therefore, isolation of this class of equipment is also important.

Equipment should be located such that the length of connecting lines is minimized. The probability of failure of a connecting line (fuel, chemicals, power supply, etc.) increases as its length increases. For example, one of the most critical systems, the emergency power generation system, is typically fed through a small diameter fuel line vulnerable to breakage from falling debris. The fuel tank should be located as close to the generator as possible, with the day tank adjacent to it.

Supporting Structures

Structural integrity of a structure that provides support to equipment is obviously as important as that of the equipment itself. The supporting structure must be designed to transfer the load induced on the equipment to the foundations. Cast iron legs, typically used as small tank supports, have historically proven to be weak and should be avoided. A wider equipment base will reduce the force necessary to overturn equipment. Manufacturers of heavy cast equipment such as pumps, mixer drive units and specialized sewage treatment equipment have claimed that their equipment bases are strong enough to resist any earthquake induced forces. These claims, however, have usually not been substantiated by actual tests or calculations.

Equipment Weight, Geometry and Response

The weight of the equipment structure (functional equipment and supporting structure) should be minimized to reduce induced earthquake loadings. Equipment structures should be as simple as possible in both plan and profile, limiting discontinuities which may allow local stress concentrations. Symmetry of the equipment structure in plan and profile minimizes its torsional response. Structures with low centers of gravity have small earthquake induced overturning movement on the base.

The equipment structure's natural response frequency should be as high as possible, above 10 to 20 cycles per second, with a minimum of 3 cps (55). As the natural frequency decreases below 33 cycles per second, the effective seismic acceleration increases. The natural frequency can be increased as follows:

a. lowering the center of gravity
b. stiffening the structure to make it as rigid as possible (e.g., adding cross bracing)
c. limiting design deformation
d. using short stocky structures if possible
e. avoiding tall, slender structures

The response acceleration to which an equipment structure is subjected is usually decreased if energy is absorbed within the structure, i.e., by increasing the damping. This can be accomplished by allowing plastic deformation of the structural materials or allowing sliding of friction joints. However, plastic deformation, yielding of the material, should be avoided for design loadings unless it will not affect the operation of the equipment or the deformed member can be quickly replaced, e.g., a mechanical fuse. Belleville washers have been used to absorb energy in mounting connections.

The structural materials should have reserve ductility. When the material reaches its yield strength, it should be able to deform without significant loss of strength. Equipment structures should be designed to resist the "design" earthquake loading without yielding. However, because of the possibility of having an earthquake inducing responses higher than the "design" level, considerations should be given to motions greater than those produced by the "design" loading. This is done by providing ductile design and details. The various structural components should be designed to yield before the structural connections, making use of the member's ductility. Stresses in structural materials should be designed to be less at connections than in the other portions of the member (170).

The earthquake induced forces on equipment components are produced by earthquake deformations. Rotating components with close tolerances may be susceptible to damage when deformed during operation. Motors, pumps, blowers, and mechanical electronic switching components are typical examples of the types of equipment that may be affected. This effect may be checked using analytical methods or shake table tests. These approaches will be discussed later in this section. Deflection of pump drive shafts with long spans extending from the motor floor to the pump floor of a pump station may be significant, causing the shaft to be out of balance. This may cause vibration and possible failure of the support bearings. This should be considered in the design of drive shaft installations.

Rigid Equipment Anchorage

All equipment should be positively anchored to resist earthquake induced horizontal forces and overturning moments. Resistance to these loadings from friction alone should not be allowed. Every attempt should be made to provide rigid anchorage, e.g., using anchor bolts set directly in the concrete or steel rather than providing resilient anchorage using vibration isolation systems. Vibration isolation systems have historically not performed well when subjected to earthquake motions. They are primarily used to isolate equipment operation vibrations from the supporting structure. They can often be eliminated by locating the vibrating equipment away from "quiet" working areas such as offices and laboratories.

Anchor bolt embedments or expansion bolts should be designed to resist the loadings calculated in Chapter VII without yielding. However, because the design levels used in earthquake design are not the maximum that may be expected, the motions experienced may exceed those calculated. To accommodate these possible increased motions, the anchor bolt steel should be designed to yield at a loading greater than the design load absorbing energy. A ductile material should be used, i.e., not cast iron. The anchor bolt steel should be designed to yield prior to failure of the concrete embedment or critical equipment elements.

Another approach is to use energy absorbing washers that deform in the equipment anchorage system. This will reduce the energy transferred to the equipment.

When expansion type anchors are used, care should be taken in drilling the holes and installing anchors. Oversized holes may result from the use of worn bits, which may not allow the specified strength of the connection to be developed. Self-drilling anchor bolt systems are recommended for this reason (171).

When shims are used to level equipment, they should provide full vertical support to the equipment base as it was designed. Failure to provide full support may allow bending of the base around the shim, allowing rocking of the structure. Stiffening of equipment bases that are not fully supported should be considered so that vibration response would not be modified by a flexible attachment (172).

Epoxy has been tested for use in equipment anchorage ("glueing" the equipment to the concrete) but has failed, as the concrete laitance layer (surface) separated from the concrete. However, epoxy has been used successfully for such items as bolt setting.

The American Concrete Institute has code requirements for design and anchorage of steel embedments as part of the "Code Requirements for Nuclear Safety Related Concrete Structures" (ACI-349) (173). A summary of these requirements is included herein and may be obtained in their entirety from ACI.

The pullout strength of the concrete is based on a uniform tensile stress of $2.6\sqrt{f_c'}$ (strength design) acting on an effective area defined as the projected area of a stress cone, as shown in Figure VI-20. The projected area is limited by cones overlapping with one another, intersection with concrete surfaces (side and bottom), and bearing area of the anchor heads. Minimum side cover distance is limited in the ACI criteria, being approximately 4 times the anchor bolt diameter for tension considerations and 10 times the anchor bolt diameter for shear considerations. Reinforcing must be added if the requirements cannot be met by the concrete. Expansion anchors are also included in the ACI criteria. Calculation of the effective area is slightly different as indicated in Figure VI-20. The minimum side cover distances for expansion anchors are about twice those of embedded anchors. When a single expansion anchor is used, a factor of safety of 2 should be applied to its design resistance. A random sample of expansion anchors should be tested to 100 percent of the design load (173).

When large shear forces are encountered in the design of anchors, they may be transferred to the concrete most effectively by the use of diagonal reinforcing bars (174). Diagonal anchors have been considered to reduce concrete cracking from shear forces being transferred to the concrete (174). This approach is, however, not commonly used.

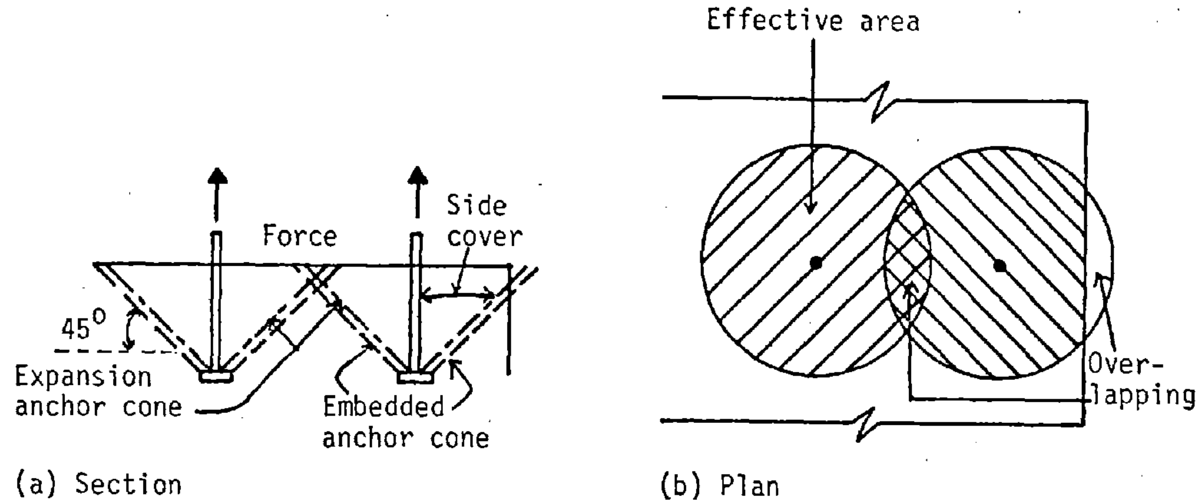


Figure VI-20. Concrete embedment.

Vibration Isolation Systems

Equipment is resiliently mounted (using vibration isolation system mounting) to filter high frequency normal operating vibrations common in rotating equipment. Vibration isolators may consist of rubber pads, laminated rubber and metal pads, single and multi-spring systems, or rubber air bags. The high frequency vibrations are filtered in the flexible isolator, which supports the structure. The resulting system has a lower natural frequency and is therefore usually subjecting the system to amplified earthquake accelerations. Because of the flexibility of the system, it usually has limited strength for resisting earthquake induced motions. The vibration isolation system may become the weak link in the equipment's support and anchorage system.

Both sides of the vibration isolation system should be properly anchored to the equipment and floor, respectively. The mass of the isolated system should be minimized to reduce the induced inertial loadings unless the system is otherwise restrained. This situation is particularly applicable to equipment bases with inertia blocks. Molded neoprene isolators have historically resisted earthquake motions better than other types of systems (088).

The best way to mitigate damage to vibration isolation systems is to make them respond as rigidly anchored systems when subjected to earthquake motions. This can be done by installing snubbers or restraints to limit the displacement to that normally encountered during the operating modes. A positive seismic activated locking device that will lock out the isolation system during an earthquake may also be used.

Figures VI-21, 22, 23, 24, 25 and 26 show typical vibration isolation system mounts.

Equipment should be mounted and operated before restraints or snubbers are installed to assure that there is adequate clearance for normal operating vibrations.

Vibration isolation systems have been used for entire floors, rooms, or buildings that contain equipment vulnerable to seismic induced loadings. The entire structure is mounted on a "flexible" isolator which increases the structure's natural period above those dominant in the earthquake's spectra. The "flexible" isolator may consist of springs or laminated rubber pads mounted between the structure and its foundation. Dampers are added to reduce the amplitude and displacement of the isolated structure. Dampers may be of the viscous type, Coulomb type (plastic deformation on bending or torsion of steel beams), or lead extrusion type. Shear pins may be used in the system to resist small disturbances (175). While isolated structures are not common, they may become useful in the housing of delicate, expensive equipment such as computers or sophisticated laboratory instrumentation vul-

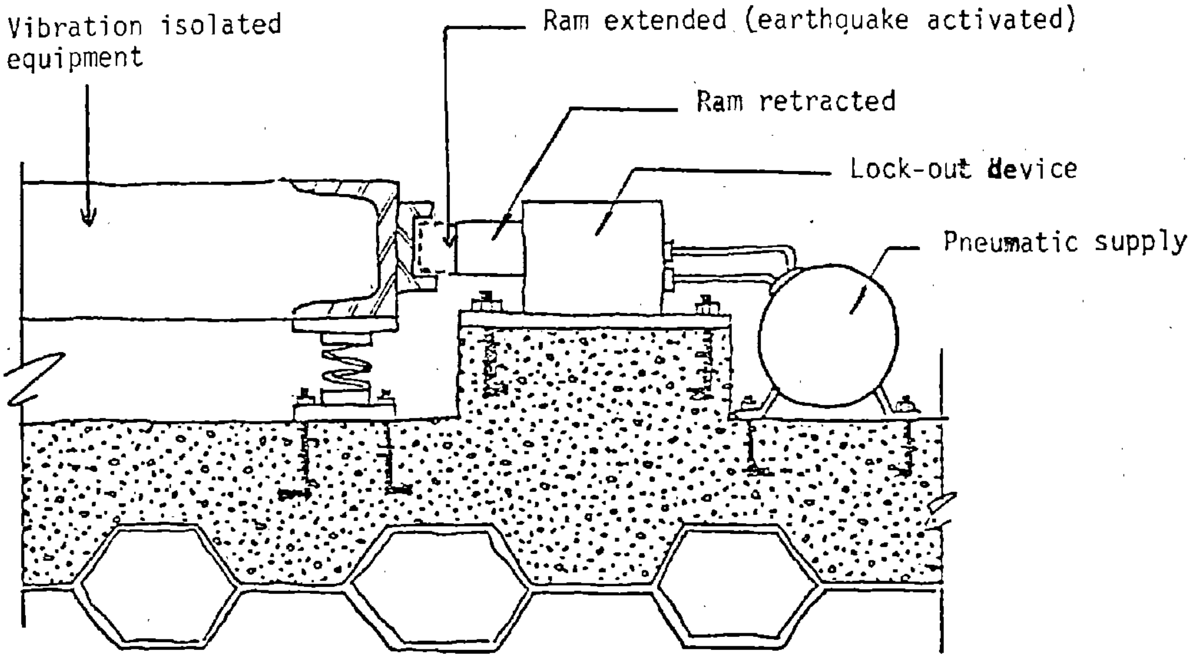


Figure VI-21. Lock-out device (176).

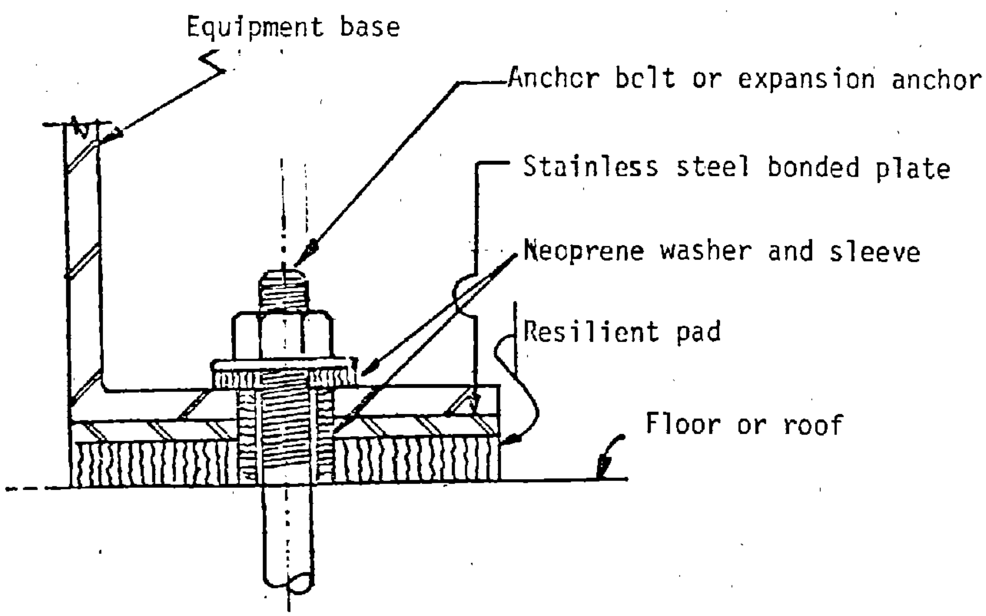
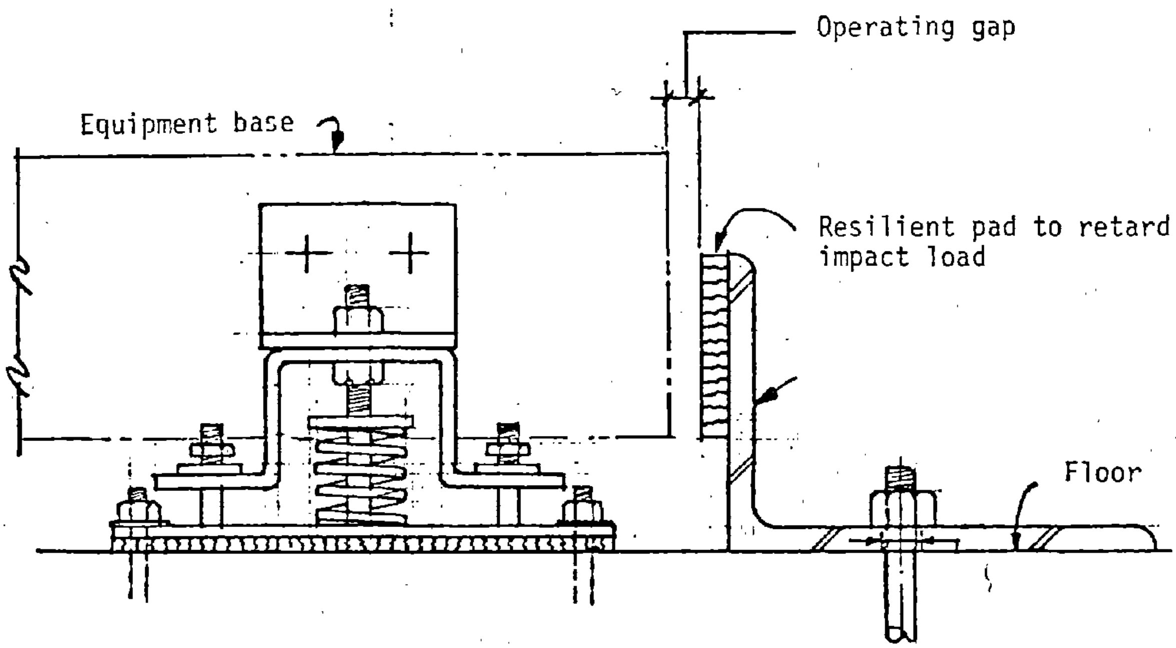


Figure VI-22. Equipment restrained by resilient pads or neoprene isolators (177).



Note: Install lateral restraining devices on all sides of the equipment base.

Figure VI-23. Restraining device for lateral loads only (177).

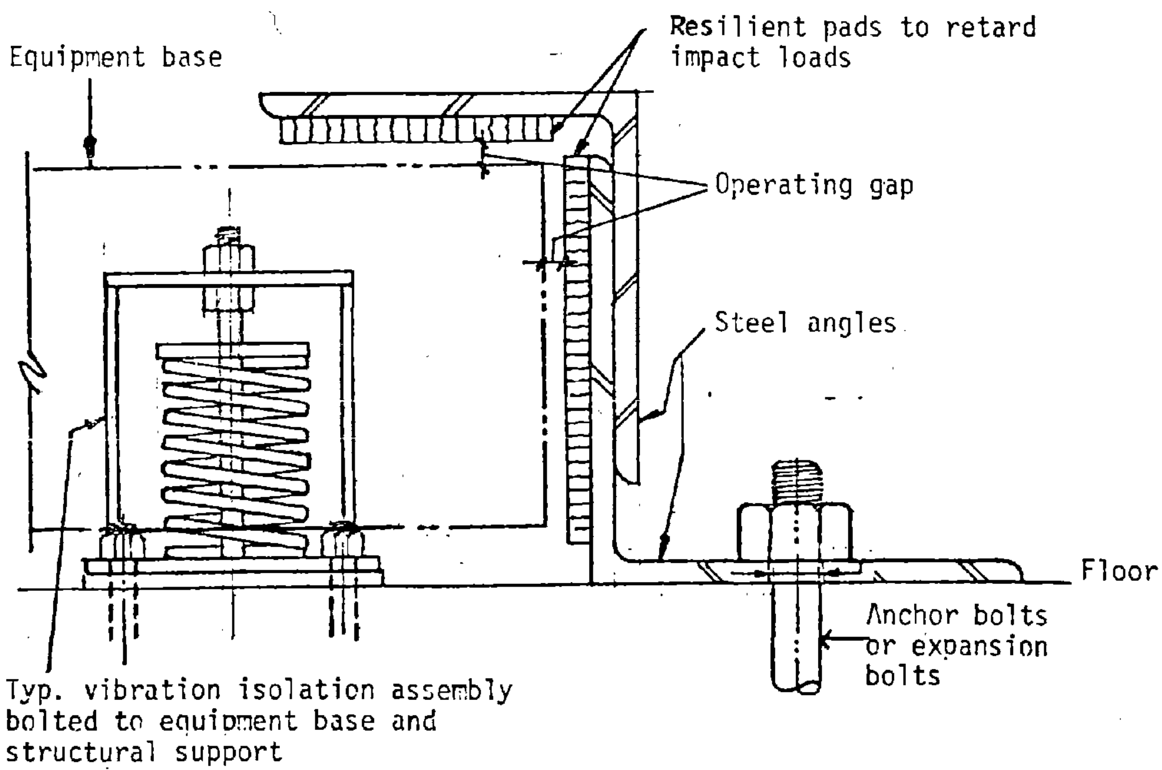
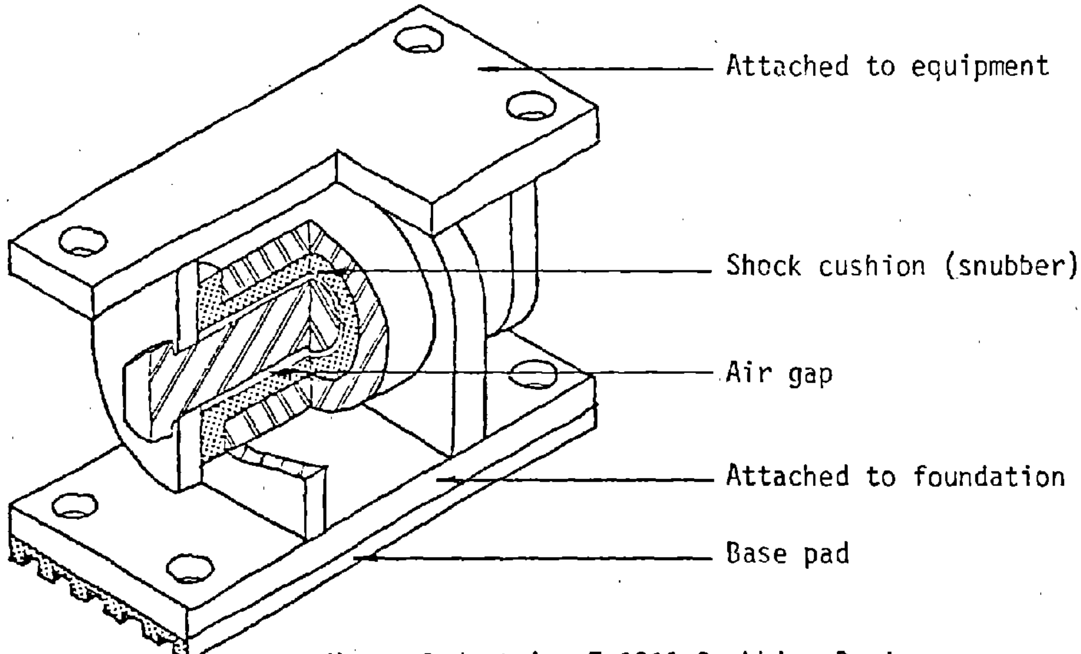
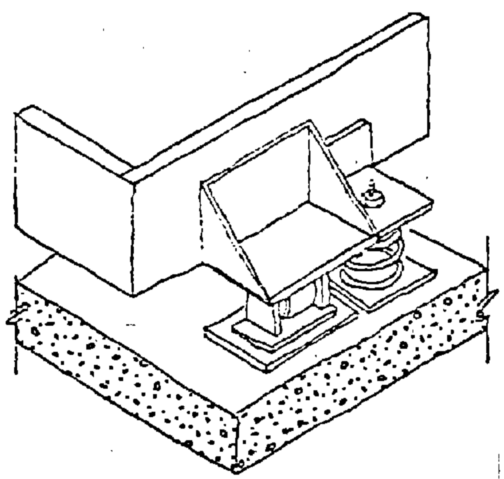


Figure VI-24. Restraining devices for lateral and vertical loads (177).



a. Mason Industries Z-1011 Snubbing Device

b. Installed under equipment



c. Installed alongside equipment

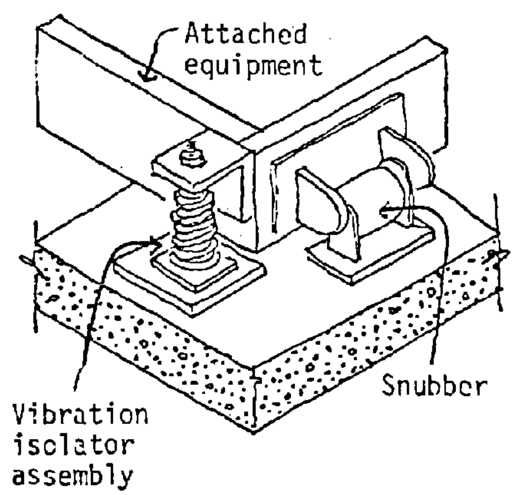


Figure VI-25. Snubber with typical installations (176).

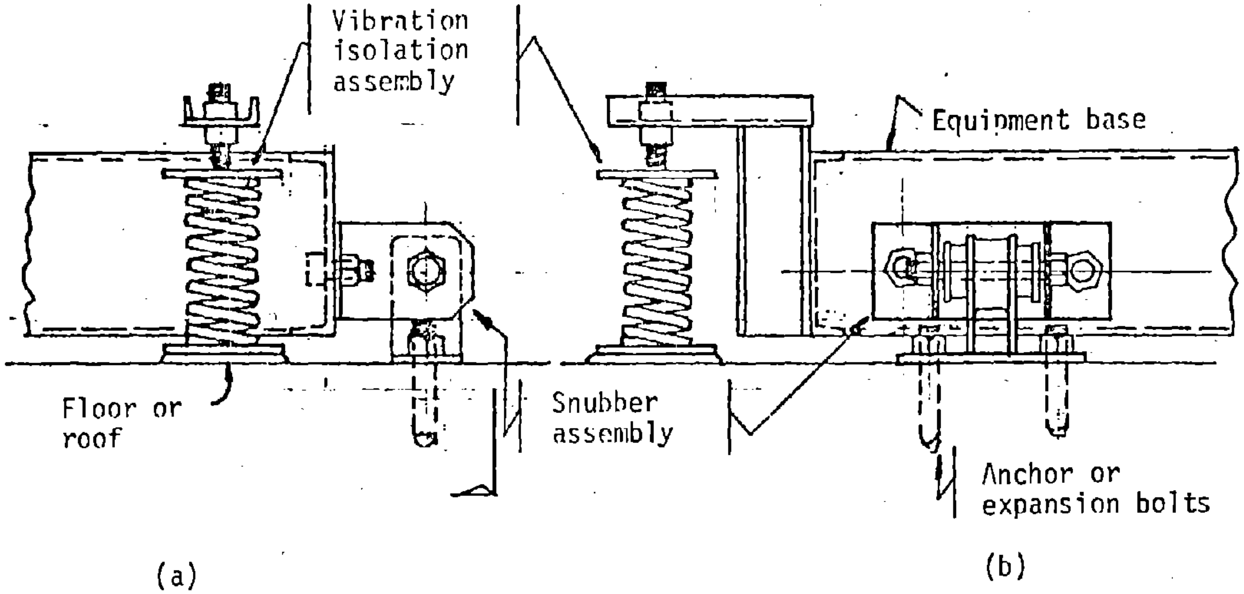


Figure VI-26. Equipment restrained by snubbers (177).

nerable to seismic motion; these may be found in modern treatment facilities.

Equipment Connections

Connections between equipment and supply systems that independently respond to earthquake motions require flexible connections. The following are examples of such types of equipment installations:

a. Between equipment on different foundations

b. Between equipment on the same foundation but with significant independent vibrations.

c. Between equipment and piping unless the pipe is short and rigidly supported.

d. Between equipment mounted on a vibration isolation system and all connecting systems.

e. Between equipment and feed lines mounted on structures not responding with the floor (interior partitions, or non rigid type construction)

f. Between systems mounted on both sides of a building construction joint.

There are very few instances where rigid interconnections should be used. One example is between two pieces of rigid, rigidly mounted equipment sharing a common foundation and capable of common vibration response.

Flexible connections for piping will be discussed in the following section. Other types of connections allowing flexibility include flexible conduit, flexible tubing, flexible canvas or rubber sections of duct work, slip joints, and mounting clearances.

Some examples of the types of interconnections recommended for specific installations are as follows:

a. Horizontal pumps with motors connected by a drive shaft should be mounted on a common foundation.

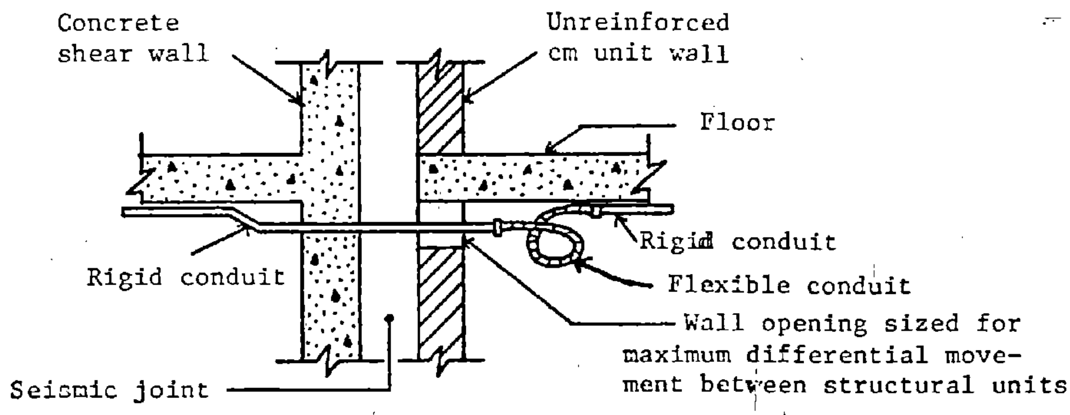
b. Where vertical pumps are driven by a drive shaft powered by a motor on a motor floor some distance above, the entire installation including the supporting structures should be rigid and respond as a single unit.

c. From a seismic response standpoint, close coupled pump-drive units are better than those supported by separate structures which may allow relative displacement when responding to seismic motion.

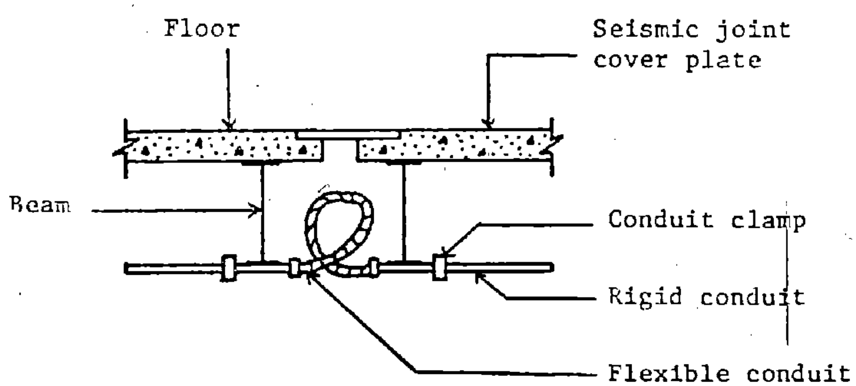
d. All piping connections to equipment including pumps, sludge processing equipment, large meters, or valves having response characteristics different from the piping and tanks should be flexible. Differential movement of rigidly interconnected equipment may cause a stress buildup in their connections, causing their failure. An exception would be for short piping interconnections between two rigid pieces of equipment on a common foundation.

e. Small diameter feed lines such as fuel lines for emergency power generators, electrical conduits, and instrumentation lines should be flexible enough to respond with the structures to which they are attached and to accommodate differential movement between adjoining structures to which they are attached. Small diameter lines crossing flexible building joints and flexible equipment mounting interfaces should allow for that flexibility in their design, as shown in Figure VI-27a and b.

f. Critical, small diameter lines such as emergency generator fuel supply lines should be encased in a conduit to protect them from falling debris during an earthquake.



(a) Suggested corrective measure for conduit crossing seismic joint.



(b) Suggested detail for conduit crossing seismic joint.

Figure VI-27. Seismic joint conduit crossing (086).

g. Connections to equipment supported by vibration isolation systems must be flexible. However, adequate flexibility must be provided to allow for the maximum displacement that may occur during an earthquake, and not merely that resulting from the design forces.

Secondary Systems

Sustained operation of most pieces of equipment rely not only on their own structural integrity, but on that of the secondary systems servicing the equipment. Most pieces of equipment depend on an electrical power supply. A subsection on emergency power supply is presented later in this chapter.

The following list contains a number of secondary systems commonly required to operate most equipment elements:

a. Electrical power supply (most equipment)
b. Cooling water (internal combustion engines)
c. Control instrumentation
d. Fuel (emergency power engines, kilns, furnaces, engine driven pumps anaerobic digestors)
e. Lubrication oil
f. Lubrication water (pump seals)
g. Exhaust (internal combustion engines)
h. Engine starting system (compressed air or battery power storage)

Because these secondary systems are as critical as the primary piece of equipment they service, the same level of seismic resistant design should be used.

Equipment Qualification and Testing

Concrete and steel structures can be designed to resist earthquake induced loadings that are calculated using the procedures in this report.

The codes governing the calculation of resistances supplied by concrete and steel members are based on many years of testing, theoretical analysis and experience. The suggested seismic code presented in Chapter VII is based on assumptions simplifying the actual response of a structure to earthquake motions. The design of equipment, however, is based on theoretical and empirical analysis of static and operational dynamic loadings as well as experience.

Seismic design of the equipment itself is a relatively new field currently being applied primarily in the nuclear industry. Attempts to calculate the responses of equipment have met with limited success due to, among other things, structural complexity. The nuclear industry requires that system components be highly reliable and requires assurance that these components will not fail when subjected to seismic motions. They have, therefore, turned to seismic qualification of the equipment for this assurance.

Many types of equipment found in the nuclear industry are similar to those found in water and sewage facilities including pumps, motors, valves, meters, and electronic instrumentation and control systems. It is suggested that the water and sewer industry develop an equipment seismic qualification procedure similar to that used by the nuclear industry. A discussion of this procedure is presented below.

The system designer must determine which equipment should have assurance provided. This assurance should be demonstrated for critical equipment necessary to meet post earthquake operation goals (up to 2 weeks). In addition, because of the availability of replacement equipment, consideration should be given to requiring some level of assurance for equipment that will be required for the system function up to 6 months after the earthquake. Some assurance can be obtained by requiring demonstrable equipment seismic qualification.

Depending on the level of qualification required, it may be demonstrated by one of the following approaches (176):

a. Dynamic shake table tests

b. Mathematical analysis

- Dynamic

- Equivalent static coefficient

c. Past experience

d. System design team judgment

e. Any combination of the above

The dynamic shake table tests are designed to subject the equipment to the conditions that would be experienced during an earthquake. The equipment would be installed on the shake table exactly as it would be installed in the facility, i.e., same anchorage, connections, operating mode, etc.

The response spectra, the motion of the shaking table simulating the earthquake motion, would be specified by the system designer. The nuclear industry subjects the equipment to the response spectra of the floor to which the equipment is anchored, usually the ground response spectra amplified at the building's natural frequencies. This approach should be considered in the water and sewer industry for use in the areas of high seismic risk. Response spectra for specific sites are not available in most cases and must therefore be developed. It is suggested that standardized response spectra be adopted by the water and sewer industry to be used for qualification of equipment. The response spectra could be adopted for the highest seismic zone with reduced percentages of the amplitudes of acceleration, velocity and displacement used for lower seismic areas.

Shake table tests may be carried out using two different approaches. One approach is fragility testing, which involves testing equipment by increasing the input motion to which the equipment is subjected until the equipment fails. Both single and multiple frequency waveforms may be used for testing. The fragility testing approach would allow qualification of equipment at all levels below the level at which failure occurred, taking some factor of safety into account.

The second approach is proof testing; each piece of equipment is tested at a specified shaking intensity. This approach would, in the long run, seem to be more expensive with no advantages for standard equipment.

Mathematical analysis of equipment provides a lower level of assurance and is recommended only for less critical equipment, equipment with simple geometry that can be easily analyzed and in areas of low seismic risk. Mathematical analysis will be most useful in determining the structural integrity of a piece of equipment, i.e., overturning, shear failure, etc. Pumps and motors have been analyzed by the use of a three-dimensional computer model checking motor rotor clearance, flange stresses, impeller casing clearance, shaft stresses and deflections, bearing loads, and anchor bolts (178). It will be of little use in determining reliability of the equipment's functioning capability, i.e., switch clearances, etc.

Qualification by past experience is essentially based on past shake table testing or mathematical analysis documentation. Qualification by the judgment of the system design team is based on inspection of the equipment by the team, when the team has experience with the response of similar types of equipment to shake table tests.

Presentation of a detailed equipment qualification procedure is beyond the scope of this report. The Institute of Electrical and Electronics Engineers, however, has developed the "IEEE Recommended Practices for Seismic

Qualification of Class IE Equipment for Nuclear Power Generating Plants" (179), which may be used as a reference.

Shake table testing is a valuable tool in developing earthquake resistant equipment. It may indicate design weaknesses which can be corrected inexpensively and which might otherwise have been overlooked. For example, equipment shown to have a low frequency could be stiffened with the addition of cross bracing. Conventional anchorage of electrical components inside an enclosure has been shown to be weak but can be reinforced for a nominal cost.

Equipment qualification poses several problems. There is some cost involved in testing and analysis. This will have a greater impact on small manufacturers. The cost of testing one equipment model will be the same regardless of whether one or many are sold. Some equipment may prove to be technically and economically impractical to test, e.g., a massive pump/motor combination. In many instances, a piece of equipment may be supplied as component parts from several manufacturers, in which case the responsibility of qualification would be unclear (090). A detailed set of specifications and a high degree of interaction between the vendor and buyer would be required to resolve this problem in an orderly manner.

DESIGN CONSIDERATIONS FOR SPECIFIC EQUIPMENT TYPES

This subsection is organized on the basis of equipment structural characteristics. The equipment listed in each category are presented as examples; the lists are by no means complete. Specific categories of equipment typically found in water and sewage facilities will be discussed, including heavy cast equipment, small tanks, cranes, electrical equipment, sheet metal structures, emergency power supplies, storage facilities and laboratory equipment. These examples will enable the system design engineer to understand the concept of seismic resistant design for each equipment category.

Cast/Heavy Frame Equipment

Examples:

a. Pumps - vertical, horizontal, submersible, detached, close connected, water, sewer

b. Blowers - centrifugal, positive displacement

c. Flocculator/Mixer/Aerator (platform mounted) drive units

d. Centrifuges - sludge dewatering

e. Motors

Considerations:

a. Supply rigid anchorage to resist shear and overturning, e.g., anchor bolts.

b. Inherently have high natural response frequency.

c. Provide flexible fittings at all pipe connections to limit stress build-up at flanges.

d. Deflection of rotating components during earthquakes should be checked (mathematical analysis).

e. Stresses on bases and long narrow appendages should be checked (mathematical analysis): e.g., vertical turbine pumps have large moments induced at their bases resulting from the weight of the pump bowls below. Vertical pumps may be supporting a motor inducing moments in the pump structure and base.

f. Deflections in drive shafts should be checked.

g. Consider mechanical seals (e.g., in potable water pumps), as earthquake damage may allow entrance of sand particles.

h. Consider a secondary power source for critical pumps (emergency power supply).

i. Insure that the seal lubrication system (grease, sealing water, etc.) is compatible with the level of function required by the equipment.

j. Low voltage and single phase protection should be provided.

k. Provide for increased loadings from hydraulic forces on the immersed portions of mixers, flocculators, and aerators.

Miscellaneous Tanks and Small Tank Like Structures

Examples:

a. Mixing tanks-steel, Fibreglas, FRP, chemical, polymer, sludge

b. Pressure filters

c. Carbon adsorption columns

d. Chemical storage tanks

e. Heat exchangers (anaerobic sludge digestors)

f. Ammonia stripping columns

g. Hot water tanks

h. Water storage pressure tanks

Considerations:

a. Supply rigid anchorage to resist shear and overturning, i.e., anchor bolts or bracing.

b. In structural analysis, assume the tank's maximum contents will respond with the tank.

c. Check the buckling stress developed in the tank shell from a combination of overturning and vertical accelerations.

d. Damping is extremely low in tall thin tanks (180).

e. Avoid structural discontinuities, (platforms, etc.) as they may allow stress concentrations and effect the structures' dynamic response (180).

f. Attached piping should include a flexible connection near the joint.

g. Brittle support legs, e.g., cast iron, should be avoided.

h. Straps restraining tanks on saddles should be welded to the tank as well as bolted to the supporting saddle.

Frame/Sheet Metal Structures (Not Including Contents)

Examples:

a. Dry chemical feeders, hoppers and storage bins

b. Cabinetry of chlorinator and liquid chemical feed systems, residual analyzers, etc.

c. Instrumentation cabinetry.

d. Lab cabinets.

e. Equipment and control consoles.

Considerations:

a. Supply rigid anchorage.

b. These structures may be supporting large masses, such as chemicals or electrical components, which induce large forces under earthquake conditions.

c. Sheet metal sides act as diaphragms, transferring induced shear to the floor. Steel/sheet metal joints must be strong enough to transfer loading. Add screws or welds as required.

d. Shake table testing may indicate weaknesses in the design; a resonance search is recommended (181).

e. Rigidity of the structure should be maximized using cross bracing, etc.

f. Locate masses as low in the structure as possible.

g. Structures with doors or removable access panels should be analyzed, as the door joint will generally be incapable of transferring a load. Multi-latch closures may be used to provide structural continuity across the joint.

h. Non-symmetrical structural characteristics (such as with doors on one side) should be analyzed, as these may allow failure of the structure in torsion.

i. Positive cabinet and file latches rather than magnetic or friction closures are recommended to resist seismic motion.

j. Laboratory cabinets may be lined with rubber mats to resist glassware breakage.

k. Locate equipment such that it is not vulnerable to failure of nearby equipment or structures.

Precision Equipment. Electronic Instrumentation and Controls

Examples:

a. Chlorinators including residual analyzers, recorders, indicators, etc.

b. Meter electronic instrumentation

c. Electronic switching gear

d. Equipment instrumentation

e. Communications systems

Considerations:

a. This type of equipment should be mounted as rigidly as possible to avoid amplification of seismic accelerations.

b. This type of equipment is a prime candidate for shake table testing and qualification due to its structural complexity.

c. Positive locking devices should be used to hold circuit boards in place.

d. All mechanical switching components, such as relays, etc., should be tested for their seismic response characteristics. Mercury switches should be avoided. Caution should be exercized in the use of gravity or light spring controlled switches. Relays have responded adequately in the energized position but have failed in the non-energized position.

e. Caution should be exercized when using friction restrained switches and components.

f. Avoid the use of circuit board mounting on stand offs, as it may result in "oil can" flexing. Use additional strengthening such as welded supports (181).

g. Communication equipment and instrumentation controlling critical equipment should be provided with an emergency power supply, possibly batteries, as well as the plant standby power supply.

h. All automatic control systems should have manual overrides.

i. Critical installations that cannot be designed to withstand seismic motion may be supported on a floor vibration isolation system designed to attenuate seismic motions.

Emergency Power Supplies

Examples:

a. Batteries

b. Secondary outside sources

c. Standby generators

Considerations:

a. Batteries should be restrained to resist overturning, shear, and vertical forces (see Figure VI-28 for an example). Batteries are commonly critical in disaster situations, as they may be required to provide emergency power to start generators, operate communication equipment and operate control instrumentation.

b. Secondary outside power supplies should have no common components with the primary power supply, such as the main power transformer, electric poles, etc.

c. Emergency power supplies (secondary sources and standby generators) for equipment critical in maintaining the function of a system following an earthquake should be completely separated from the primary supply (i.e., separate conduits, power panel, etc.).

d. Standby generators should, if possible, be rigidly anchored to the floor. If vibration isolation systems are used, snubbers should be employed.

e. Flexible connections should be provided for the fuel, cooling water, exhaust, and electrical systems' attachments to the generator set.

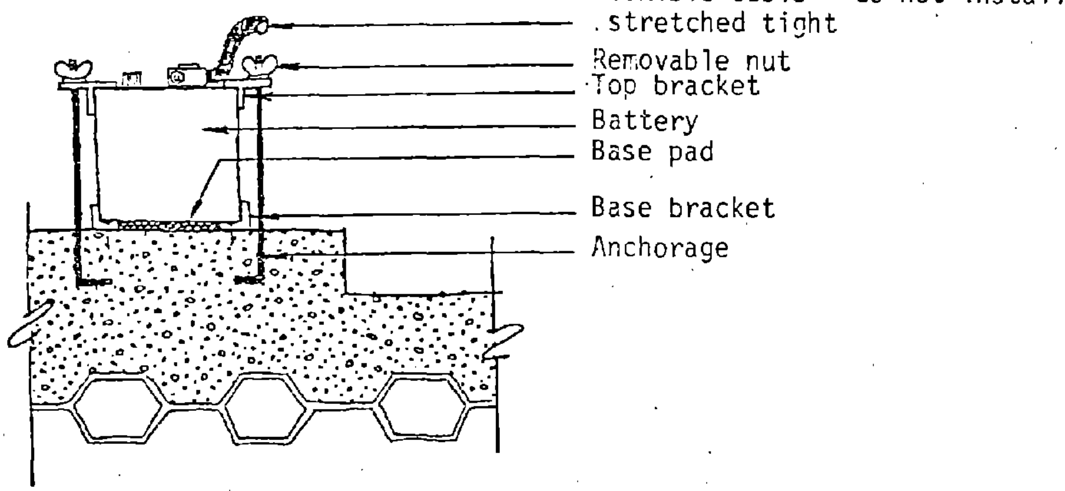
f. Service lines should be kept short.

g. Powered fuel oil pumps serving the standby generator should have a backup system for filling the day tank (possibly a manual pump) (086).

h. Fuel lines should be protected from falling debris (086).

i. Multi-fuel supply systems should be considered (086).

j. Cooling water systems should be independent of other water systems (such as potable water systems).



Note: Shock material such as styrofoam should be placed snugly beneath and between batteries. Baking soda may be sprinkled over the top of batteries to reduce corrosion.

Figure VI-28. Emergency power supply battery set (176).

Immersed Equipment

Examples:

a. Air diffusers

b. Floating aerators and impellers of platform mounted aerators

c. Flocculator paddles/impellers

d. Mixer impellers

e. Launders/collection/distribution troughs

f. Overflow weirs

g. Sludge scrapers

h. Scum skimmers

i. Baffles

j. Piping

Considerations:

a. Provide seismic shut-off switches for rotating equipment that could be affected by wave action (e.g., flocculators, mixers, aerators, sludge scrapers, scum skimmers, etc.).

b. Provide break-away mountings (mechanical fuses) for equipment that cannot be designed to withstand seismic induced hydraulic forces (e.g., floating aerators, launders, scum skimmers, baffles).

c. Limit the use of equipment with a large projected surface area immersed in water wherever possible (e.g., center feed clarifiers [baffles], reactor clarifiers [baffles], etc.)

Refractory Equipment

Examples:

a. Furnaces

b. Kilns

c. Incinerators

d. Chimneys and smoke stacks

Considerations:

a. Refractory material is typically heavy and brittle, exactly the opposite of what is recommended for seismic resistant design.

b. Refractory equipment should be designed to respond rigidly so that differential movement between components, which may cause cracking, will be limited.

Lab and Office Equipment

Examples:

a. Stills

b. Refrigerators

c. Incubators

d. Jar testing equipment

e. Typewriters

f. Analytical equipment (e.g., chromatographs, spectrographs, etc.)

Considerations:

a. All equipment should be restrained if possible. See Figures VI-29 and VI-30.

Hydraulic Flow Control and In Channel Equipment

Examples:

a. Valve controls - valve stands, hydraulic and pneumatic cylinders

b. Sluice gates and operators

c. Bar screens

d. Comminuters

e. Open channel flow measurement, weirs, flumes

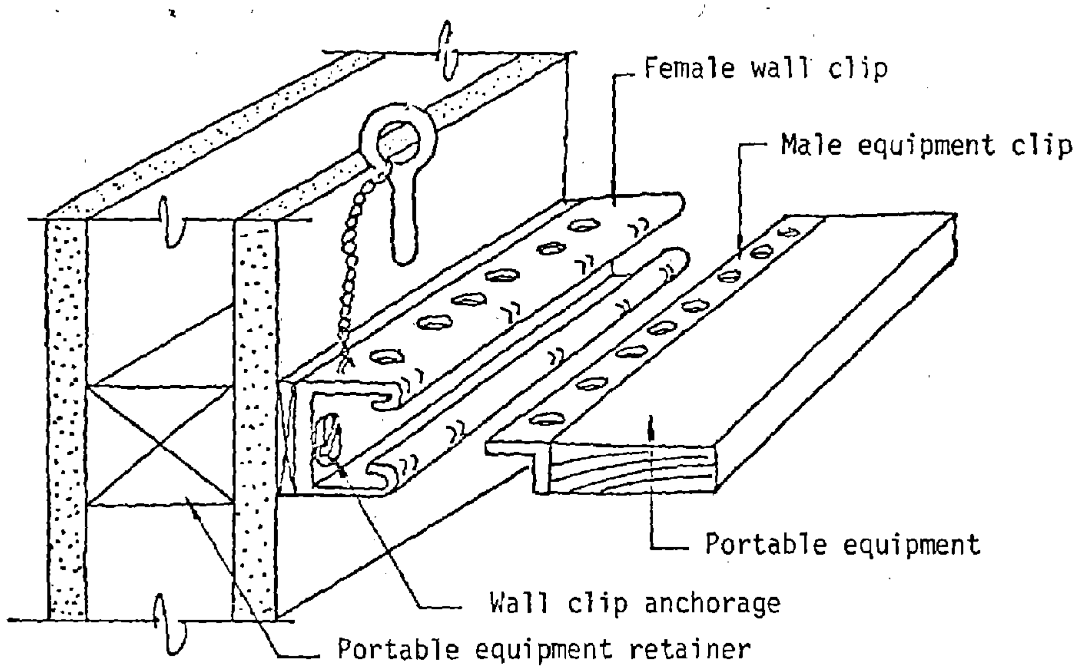
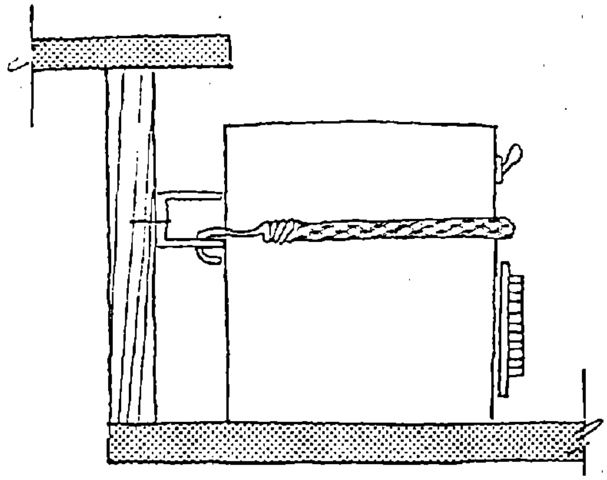
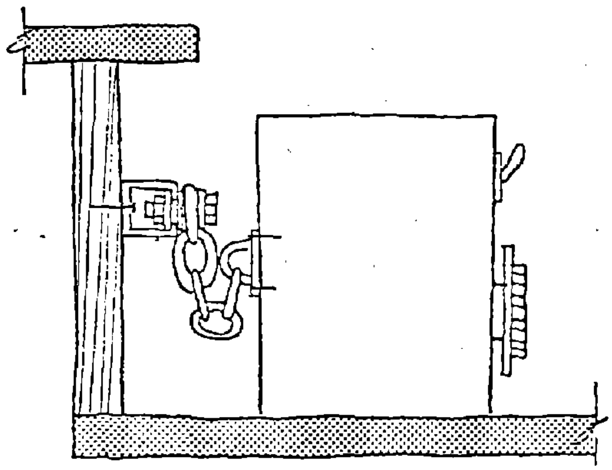


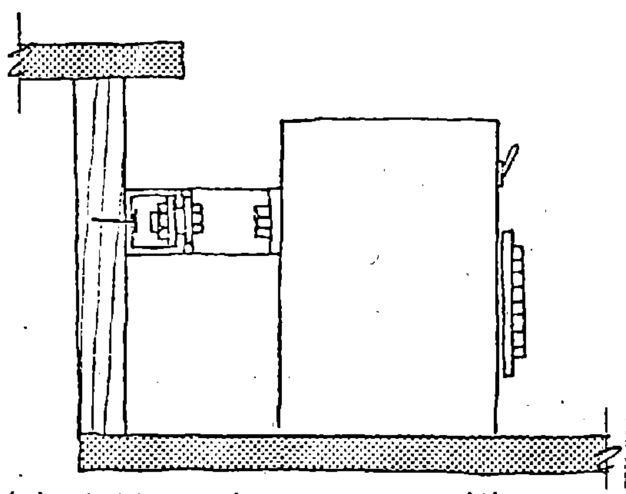
Figure VI-29. Counter top item attachment with elastic straps (176).



(a) Elastic strap to front of equipment



(b) Flexible attachment



(c) Rigid attachment at rear with spacer

Figure VI-30. Rigid counter top equipment attachment (176).

Considerations:

a. Flow controls (sluice gates, valves, etc.) are typically required to maintain operating goals following an earthquake and should be carefully designed.

b. Automated controls should have manual overrides.

c. Equipment appendages (valve stands, cylinder operators, etc.) should be analyzed as cantilevers.

d. Structures embedded in concrete (sluice gate guide rails, bar screens, etc.) will respond as an integral part of the concrete structure. The surrounding concrete should be designed to prevent cracking and displacement so that clearances required for embedded equipment operation will be maintained.

e. Hydraulic flow through the plant must be maintained following an earthquake. Therefore, bar screen and comminuters should be designed with alternate flow paths in the event of failure of one of these pieces.

Liquified Gas Storage and Handling

Examples:

a. Chlorine cylinder scales

b. Chlorine cylinder storage

c. Tank car storage

d. Chlorine cylinder connections

e. Other types of cylinder storage and handling

Considerations:

a. All references to chlorine shall include other hazardous chemicals.

b. All gas cylinders should be chained or blocked to prevent overturning or rolling (see Figure VI-31).

c. Chlorine scales should be equipped with snubbers to prevent lateral motion with positive tank anchorage to the scale.

d. Railroad tank cars should be blocked to prevent rolling.

e. Chlorine lines should be protected from falling debris.

f. Chlorinator systems should be designed to exclude pressurized chlorine gas lines (tank mounted controls). If lines are ruptured, chlorine gas leakage would be minimized.

g. Chlorine solution feed lines should be kept as short as possible.

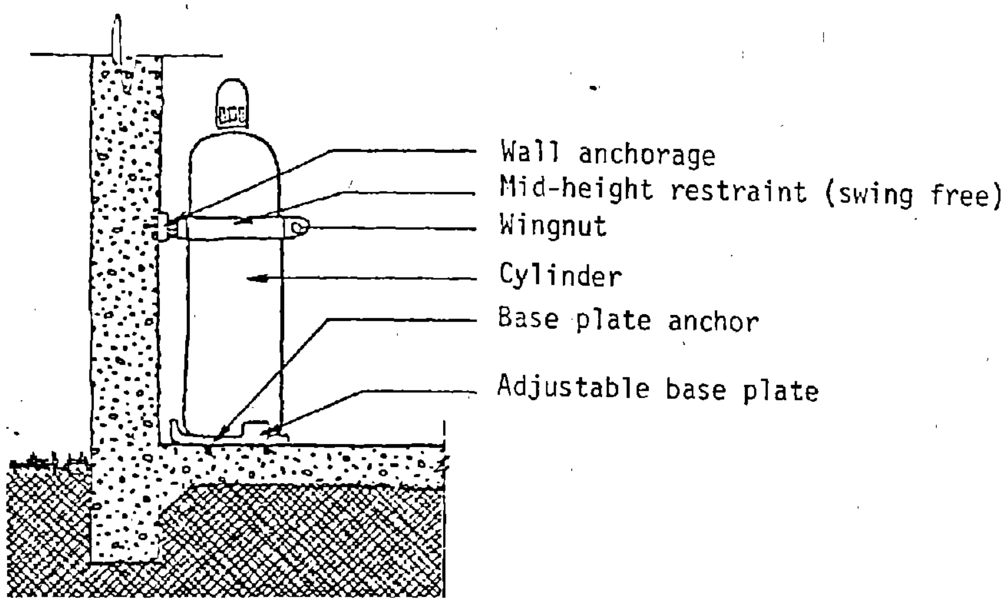


Figure VI-31. Gas cylinder installation (176).

Cranes and Hoists

Examples:

a. Chlorine tank hoists

b. Shop cranes (jib or overhead bridge cranes)

Considerations:

a. Systems should be designed to limit differential movement of the rails (particularly horizontal) that may allow derailment of the bridge.

b. Consideration may be given to providing a vertical restraining device to prevent the bridge from "bouncing" off the rails.

Pole and Pole-Mounted Structures

Examples:

a. Transformers

b. Antennas

c. Electric transmission poles

Considerations:

a. Structures should be properly designed and guyed, if necessary, to resist earthquake accelerations.

b. As a possible alternate, structures supported on poles may be designed to be mounted on the ground.

Specialized Structures

Examples:

a. Vacuum filter

b. Sludge presses

c. Air compressor/storage tanks units

d. Filter surface wash systems

e. Travelling bridge filter backwash systems

f. Travelling bridge sludge collectors

g. Conveyors

h. Trickling filter distribution systems

i. Rotating biological contactors (RBC)

Considerations:

a. This class of equipment often includes complex structures that may be too large to economically test on a shake table. Standard seismic resistant design procedures should be applied where possible.

b. Rigid anchorage should be provided.

c. Structures supporting large masses such as RBC units supporting disks require a detailed structural analysis of support members.

d. Bridge structures such as backwash and sludge collection systems should be analyzed for their dynamic response and deformation because of their long spans and simply supported end connections.

e. Systems supported on rails (e.g., travelling bridges) may be fitted with clips to prevent them from "bouncing" off the rails.

f. Consideration should be given to providing a seismic activated shut-off switch for rotating equipment that may be damaged when operated during an earthquake. Deflections of rotating components may occur. Components that suffer minor earthquake damage may cause further damage if allowed to continue operating.

PLANT PIPING

This section includes design considerations for plant piping, including exposed piping intermittently supported. Considerations for pipe material, structural design (span, deflection, etc.), hangar design, flexible joint design, wall penetrations, and pipe appurtenances are included.

Rigid pipeline systems with no flexible joints or allowance for flexure should be supported by a single structural system, one that will respond as a single structure, unless specific provisions are made for movement of supports attached to other structures. Where different structural systems are used for anchorage and relative displacement may occur, flexible joints that will allow relative movement should be provided in the piping systems. Examples include wall penetrations, anchorage to the floor with adjacent ceiling anchorage, building joint crossings, etc. Piping systems should have continuous response characteristics. Any discontinuities in the system should be separated from the system by flexible joints or other techniques. Discontinuities may include valves, meters, fittings in the line, or major changes in the pipe size (e.g., main transmission line feeding smaller branch lines). Continuity should be maintained in the type of anchorage used. Flexible supports in combination with rigid supports may allow stress buildups and failure of rigid supports. Long continuous pipe runs are seldom critical structurally.

Clearances should be maintained between pipes and adjacent pipes or structures. If a pipe is expected to respond differently from the wall through which it passes, a void should be left around the pipe. Adjacent parallel pipes should be separated a minimum of four times the maximum displacement calculated from earthquake induced forces unless spreaders are employed. A clearance of three times the maximum calculated pipe displacement from earthquake induced forces should be allowed between pipes and walls or other rigid elements, with a minimum of three inches maintained (168).

The National Fire Protection Association (NFPA) has presented criteria for seismic piping in NFPA Publication #13, Installation of Sprinkler Systems, 1978 (182). As NFPA systems have responded well to past earthquakes, fire protection systems should be designed, as a minimum, in accordance with NFPA #13. Some of the considerations included in NFPA #13 will be presented in this subsection.

This report is primarily concerned with process piping which generally has a minimum diameter of 4". Small piping is inherently more ductile with a substantially smaller mass than large pipes and is therefore less susceptible to failure from earthquake loadings. Gas piping less than 1" in I.D., boiler piping less than 1-1/4" I.D. and all other piping less than 2-1/2" in I.D. (e.g., house plumbing, sampling lines, etc.) need not be considered in seismic resistant design calculations (158). While small diameter pipes may be intrinsically more resistant to direct seismic damage, they have experienced extensive damage in past earthquakes due to failure at joints and failure of equipment or structures to which the piping is attached. However, because these small diameter lines are susceptible to damage, lines providing critical functions and those carrying hazardous chemicals should be protected from falling debris.

Pipe Materials

Plant process piping typically is constructed of steel or ductile iron in sizes larger than 4" in diameter. Fiber reinforced plastic, F.R.P., is also available with flange joints, but has gained little acceptance for conventional installations. Both steel and ductile iron pipe have ductile properties that will allow some deformation before failure, a quality sought after in seismic resistant design. In recent years, ductile iron has replaced cast iron pipe because of advancing technology. This is advantageous for seismic resistant design, as cast iron pipe is brittle and allows little differential movement before failure occurs. However, smaller ductile iron pipe fittings are still fabricated from cast iron. Care should be taken in design when they are used.

Ductile iron plant piping typically employs flange joints. Either welded, flange or screwed (small diameter) joints are used with steel pipe. Welded, screwed and flange joints are rigid, adding no flexibility to the piping systems. Welded joints may have some nominal advantage over flange joints as they provide greater structural continuity across the joint, limiting stress buildup where failure may occur. If iron or steel pipe with screw fittings are used, great care should be taken in pipe restraint design and flexibility provisions, as screw fittings have had a poor record in resisting earthquake movement.

Pipe Design

A procedure for calculating forces induced in piping systems is presented in Chapter VII. A table listing maximum unsupported pipe spans is included as part of Figure VII D-11. This table was prepared on the basis that the natural frequency of the piping system should be designed to be greater than or equal to 20 cycles per second, a theoretical condition required for a "rigid" system. Maintaining a rigid system limits the amplifications of earthquake accelerations to which the system is subjected. Design of flexible piping systems where the span is limited by pipe deflection is covered in the Tri-Service Manual (158) but is not recommended. Design earthquake induced forces may be increased by as much as a multiple of 5 using that approach.

The NFPA limits the maximum span of sprinkler piping to 15 feet for pipes larger than 1-1/2" in diameter to maintain system rigidity (182).

The nuclear industry utilizes computer programs such as STRUDEL and SAP to calculate pipe stresses. The computer programs may include a dynamic analysis. Earthquake motion inputs may be included in the computer programs. Even with the use of computer analysis, the nuclear industry requires the use of rigid piping systems. Systems are commonly designed to have a minimum natural frequency of 33 cycles per second. The Tennessee Valley Authority requires the maximum static pipe deflection to be limited to 0.009 inches as well as keeping the natural response frequency above 20 cycles per second.

It is not the intent of this report to restrict the design approach to that presented, but to provide a basic approach when others are unavailable. Any analysis which accounts for seismic induced forces at levels comparable to those presented herein is acceptable.

Pipe Hangars and Lateral Bracing

In addition to loads normally included in pipe support design, pipes should be rigidly supported to resist earthquake induced loading. This requires not only an increased vertical loading resistance, but horizontal bracing, both perpendicular (resisting lateral displacement) and parallel (axial loading), as well. Sway bracing should be provided to maintain a rigid system laterally, similar to vertical support systems. One axial brace for each run (straight length) of pipe is adequate unless extremely long runs are encountered (182). Bracing should be designed to resist the forces calculated in accordance with the procedures included in Chapter VII. Figure VI-32 shows a number of acceptable seismic details for sway bracing (158, 177).

The slenderness ratio l/r for sway bracing designed to resist forces in compression should not be greater than 200, where l is the distance between the center lines of the pinned supports and r is the smallest radius of gyration of the member as found in structural steel manuals. Table VI-5 shows maximum lenghts of shapes used for sway bracing in compression.

TABLE VI- 5 . MAXIMUM PIPE BRACE LENGTHS (182)

Item	Max. Length l/r = 200	Item	Max. Length l/r = 200
Angles		Flats	
1½ x 1½ x ¼ in.	4 ft 10 in.	1½ x ¼ in.	1 ft 2 in.
2 x 2 x ¼ in.	6 ft 6 in.	2 x ¼ in.	1 ft 2 in.
2½ x 2 x ¼ in.	7 ft 0 in.	2 x 3/8 in.	1 ft 9 in.
2½ x 2½ x ¼ in.	8 ft 2 in.	Pipe	
3 x 2½ x ¼ in.	8 ft 10 in.	1 in.	7 ft 0 in.
3 x 3 x ¼ in.	9 ft 10 in.	1¼ in.	9 ft 0 in.
Rods		1½ in.	10 ft 4 in.
3/4 in.	3 ft 1 in.	2 in.	13 ft 1 in.
7/8 in.	3 ft 7 in.		

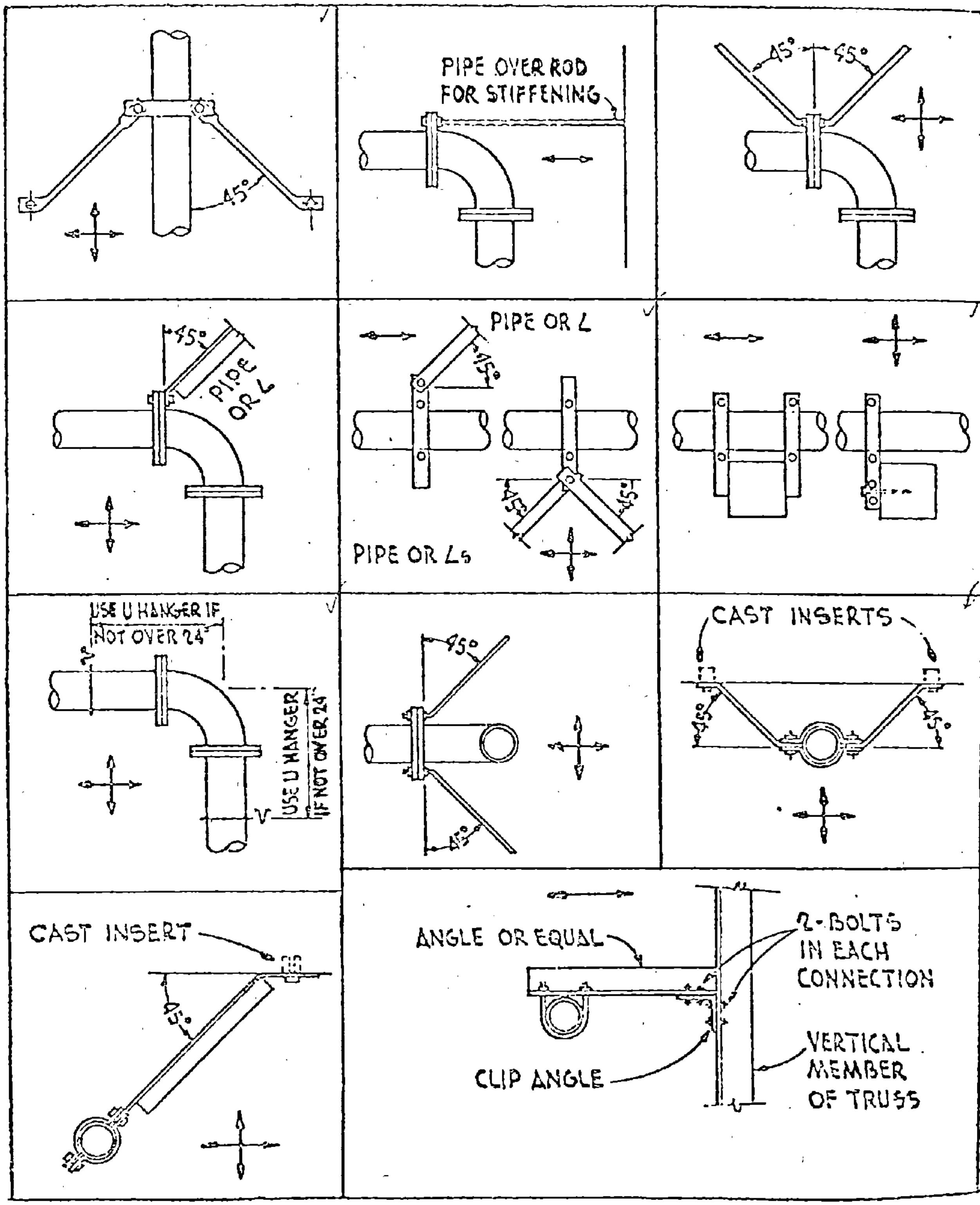


Figure VI-32. Acceptable seismic details for sway bracing (158).

There may be reason to support piping in other than rigid systems such as making allowance for thermal expansion and vibration isolation for noise reduction. Thermal expansion joints should be restrained with a hydraulic shock absorber that will allow slow thermal expansion and contraction but will respond nearly as a rigid member when subjected to earthquake motions. This type of system is commonly employed in steam lines in power plants subject to extreme temperature differentials. Vibration isolation mounting should be provided with snubbers in much the same manner as those used for equipment.

Consideration may be given to using energy absorbing restrainers such as the Ramberg-Osgood type to limit the acceleration to which the pipe is subjected (183). However, when movement is allowed at one support, comparable movement must be accounted for at all other supports of the rigid system. Rigid systems must be separated from those allowing some movement by providing a flexible connection.

In many instances, pipe hangar design has been left up to the contractor. Because hangar design details are important in seismic resistant design, the design engineer should provide hangar design details for each installation. A high level of quality control through inspection should be provided.

Flexible Joints

As previously discussed, flexibility is required between adjacent rigid systems. This flexibility can be attained by the use of flexible joints or systems including mechanical joints, sleeve type joints, rubber bellow joints, light metal bellows joints, multiple ball and socket joints, or with piping loops.

Mechanical joints - (See Figure VI-33)

a. Available for ductile and cast iron pipe.

b. Allows rotation about the pipe axis and limited axial and angular flexibility.

c. Restrains movement of joint by friction of gasket against pipe.

d. Limited flexibility compared to other types of flexible joints, although more flexible than a flange joint.

e. As the diameter increases, the maximum angular flexibility decreases.

f. Requires axial thrust restraint with thrust blocks or rods to prevent pull out of spigot from bell.

g. Relatively inexpensive and readily available.

Sleeve joints - (See Figure VI-34)

a. Available for most types of pipe.

b. Allows rotation about the pipe axis and limited axial, angular, and transverse flexibility.

c. Requires thrust restraints, as mechanism is similar to mechanical joint (Note: Restrained joint prevents pullout from earthquake movement, not restrain pressure).

d. Angular flexibility, 4 degrees per joint up to 30" pipe (7" sleeve) with reduced maximum flexibility as pipe diameter increases. Expansion joints will further limit flexibility (dependent on particular manufacturer).

e. Relatively inexpensive.

f. Allows for easy equipment installation and removal.

g. Allows dimension flexibility in installation.

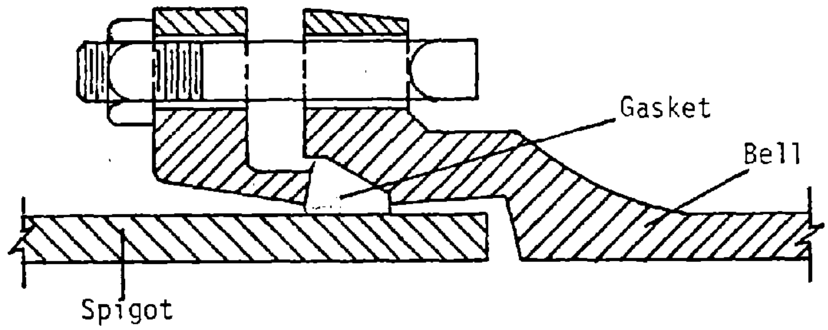
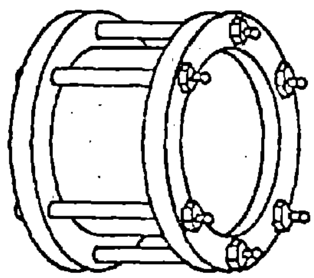
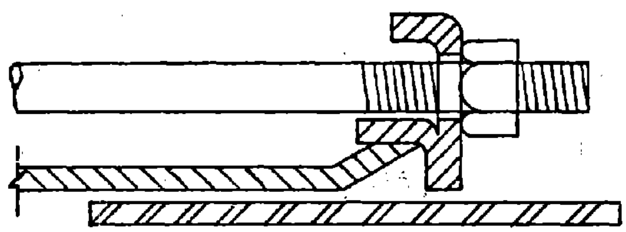


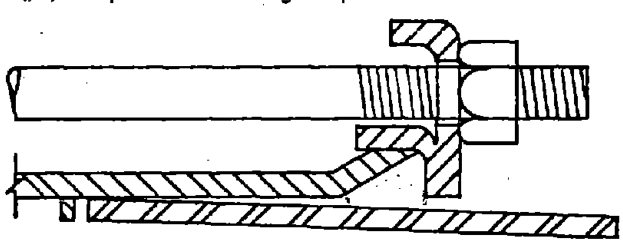
Figure VI-33. Mechanical joint (184).



(a) Typical sleeve joint



(b) Pipe in straight position



(c) Pipe in deflected position

Figure VI-34. Sleeve joints (185).

Rubber bellows - (See Figure VI-35)

a. Available for flange joint connections.

b. Requires thrust restraints (Note: The optional stretcher bolt limits the flexibility of the joint.).

c. Allows limited axial (1"-1.5"), transverse and angular movement but allows minimal rotation deformation about the axis (i.e., 3^{o} up to 12", 2^{o} up to 16", 1^{o} up to 18", depending on manufacturer).

d. As the size increases its maximum angular flexibility decreases (i.e., 12"-12^{o}; 18"-8^{o}, depending on manufacturer).

e. Flexibility of the joint is included in flexibility of the rubber, not in the slippage of the gasket on the pipe. It is therefore easier to deform and is better suited to attenuate vibration transmission of high frequency vibrations (operating mode of pumps etc.).

Metal bellows joints - (See Figure VI-36)

a. Available in flange joint connections.

b. Requires axial thrust restraints in addition to limit stops

c. Single joints designed primarily for axial displacement but, used in series, allows lateral and angular movement. Does not allow rotation about the pipe axis.

d. Experience in EBMUD and the oil refinery industry has indicated problems with corrosion of the bellows causing failure of the joint.

Ball and socket joints - (See Figure V-37)

a. Available with steel and flange joint connections.

b. Does not require thrust restraint as it is carried through the joint structure.

c. When one joint is used independently, it will allow infinite rotational flexibility about the pipe axis, and angular flexibility,

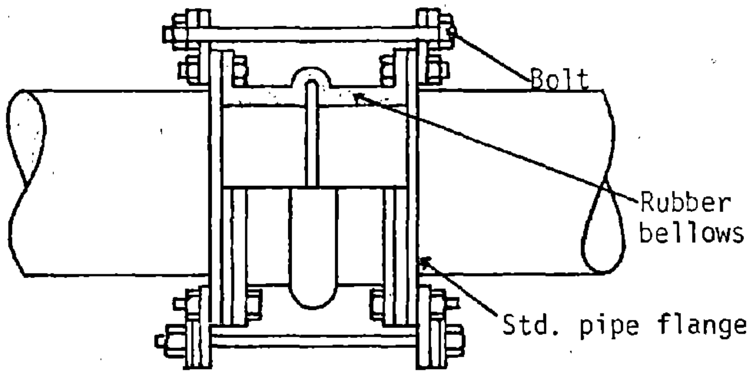


Figure VI-35. Rubber bellows (186).

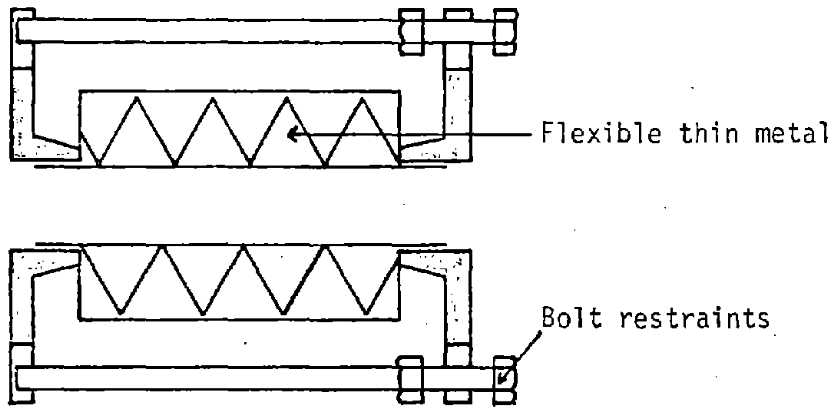


Figure VI-36. Metal bellows joint (187).

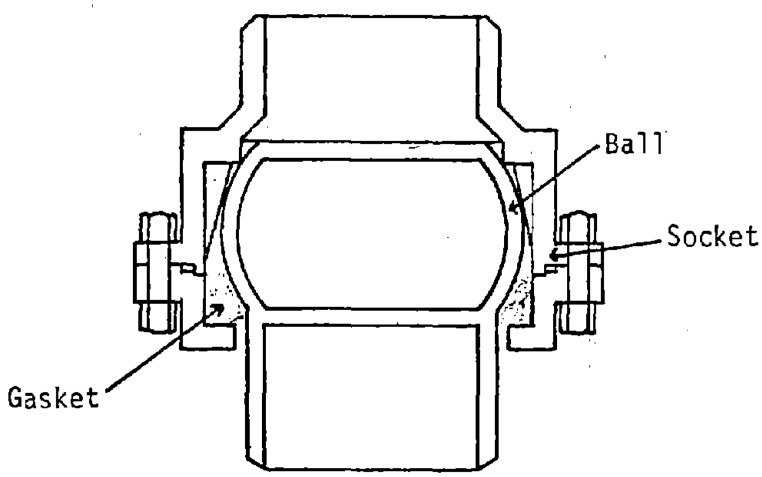


Figure VI-37. Ball and socket joint (188).

either 7-1/2° or 15° bend from the straight position, depending on the particular design. It will not allow any axial movement or lateral displacement when used independently.

d. When 3 ball and socket joints are used in series, aligned on 3 different axes, they will allow movement in all directions limited only by the length of rigid pipe connecting them.

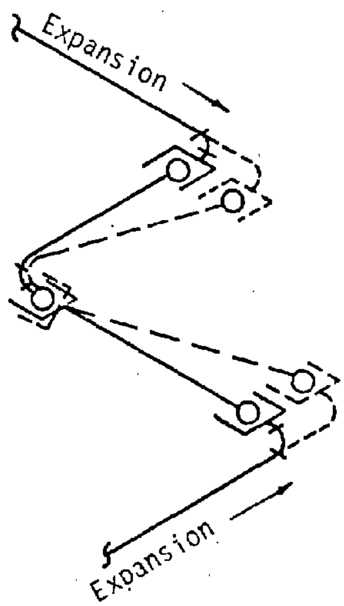


Figure VI-38. Schematic of three ball and socket joints used in series (189).

e. When the system is pressurized, the joint will lose some of its flexibility.

f. The machined surfaces may corrode, reducing the joint's flexibility or "freezing" the joint, allowing no flexibility at all. It is believed that the surfaces in a "frozen" joint would break before other components of the system would fail.

g. To the authors knowledge, there have been no large diameter pipe ball and socket installations subjected to earthquake motions.

EBMUD, however, has been installing 3 ball and socket joint installations since approximately 1976.

h. Ball and socket joints, because of their machined surfaces, are very expensive.

Pipe loops -

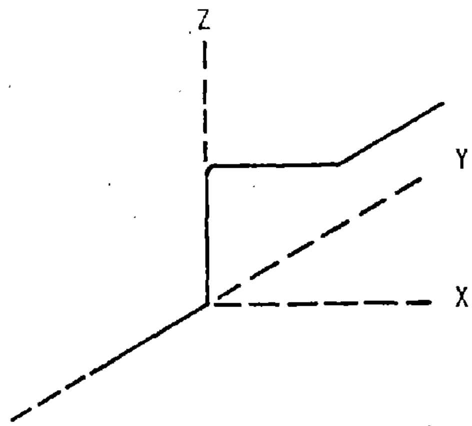


Figure VI-39. Schematic of pipe loops

a. Pipe loops make use of the elasticity of the pipe, lengthening the pipe sections on each leg to minimize the strain in the pipe.

b. Loops will provide flexibility in all directions and at any amplitudes required, depending on the pipe material, pipe layout, and the length of legs used in the loop.

c. An elastic material is required such as steel or ductile iron. Cast iron should not be used in pipe loops designed to allow flexibility.

d. Pipe loops are commonly used in the nuclear and oil industries to allow for thermal expansion and contraction.

e. The pipe material in the loop is typically the same as that of the entire piping system and is therefore not subject to corrosion or other problems in excess of those of the system itself.

f. Pipe loops require more space than other types of flexible joint systems and may therefore not be feasible, depending on the situation.

Degree of flexibility - The degree of flexibility required is based on the expected differential movement of the systems that are connected. EBMUD has arbitrarily picked a design differential movement of 6" between a value pit and buried piping. This would seem to be the maximum differential movement that would be encountered. The least differential movement between systems would occur at the connection of two rigid systems, e.g., a pump and a properly braced piping system.

Recommendations - To achieve maximum flexibility, ball and socket joints are recommended. EBMUD achieves an allowable 6" differential movement by using ball joints that can rotate 7-1/2^{o} in conjunction with a 4' pipe leg for each joint. Thin metal bellows type joints should be avoided, as these are subject to corrosion problems. Rubber bellows type joints have limited angular flexibility, particularly in larger pipe diameter. Some designs (e.g., multiple bellows), may, however, accommodate adequate flexibility.

For systems requiring allowance for small differential movement, sleeve type joints may be used effectively at a reasonable cost.

Bracing and restraints - Adequate bracing must be provided for flexible joint connections. Pipe movement relative to the pipe supporting structure must be limited to the pipe section designed to move. Pipe guides must be provided to limit the relative motion on the flexible joint to the direction for which it was designed. For example, 3 flexible joints in series may allow lateral pipe movement and failure as shown in Figure VI-40.

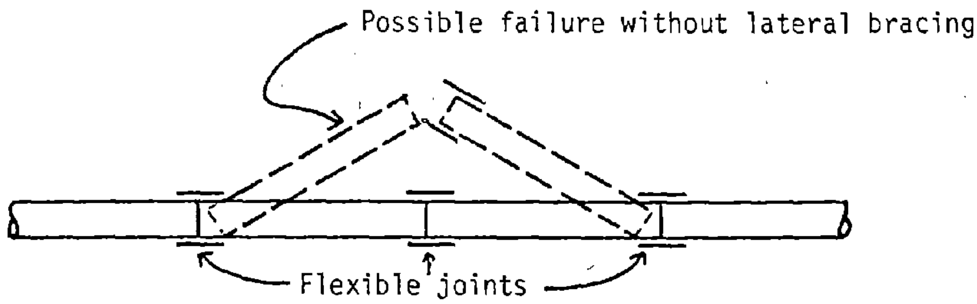


Figure VI-40. Lateral pipe movement when using three flexible joints in series.

Thrust restraints must be provided for mechanical, sleeve, metal bellows, and rubber bellows flexible joints. The internal water pressure attempts to force joints apart. In the case of joints with limit restraints, the pressure will separate the joint to the limit restraints, eliminating any axial extension flexibility for which the joint was intended. The purpose of the limits is to prevent separation of the joint when subjected to differential movement beyond the capability of the joint to absorb, not for thrust restraint. Joints with neither limits nor thrust restraints may separate shortly after the system if pressurized.

Wall Penetrations

Non-water tight wall penetrations are typically used between adjacent "dry" areas in sewage and water facilities. In these areas, adequate clearance around the pipe should be provided to allow expected movement. If a physical barrier is required between the two areas, an easily deformable material should be used. Water tight penetrations are typically found in tank walls and below grade vaults or basements. If the penetration is grouted in the wall, rigidly supported, flexible joints should be provided on both sides of the wall. If lateral displacement or shear is expected between the pipe and the wall, as may occur when a structure settles during an earthquake, two flexible joints should be provided on the side of the wall where the shear is expected; this is illustrated in Figure VI-41.

When a flexible wall penetration is provided, a single flexible joint on each side of the wall should be adequate to absorb differential movement. A flexible modular wall penetration seal that is commercially available is shown in Figure VI-42. This type of wall penetration seal was used in tank connections in EBMUD's recently completed secondary sewage treatment plant.

When large differential movements between the pipe and wall are possible, wall penetrations with greater flexibility are suggested, with a flexible joint system on the interior that will absorb greater differential movement.

Piping Appurtenances

Valves, meters, backflow preventers, strainers, air filters, and blower silencers are commonly included in water and waste facility piping systems. If these appurtenances effect the response of the piping system, they should be independently supported. If differential movement is expected between the appurtenance and the piping systems, a flexible joint should be provided.

Automatic Shut-Off Valves

Automatic shut-off valves should be provided on all hazardous material piping systems. These systems may include natural gas, chlorine gas or solutions, acid or base solutions, and methane gas (anaerobic digesters). The

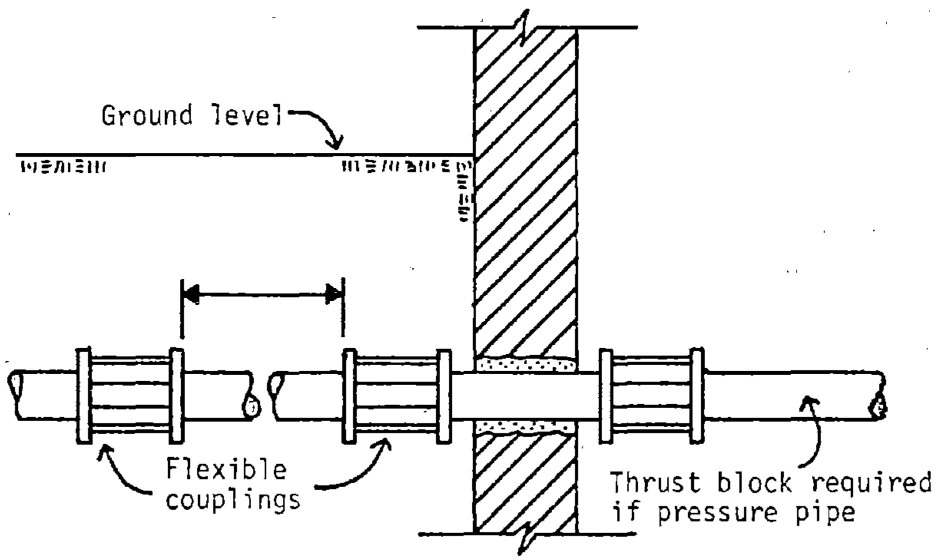


Figure VI-41. Grouted pipe wall penetration (158).

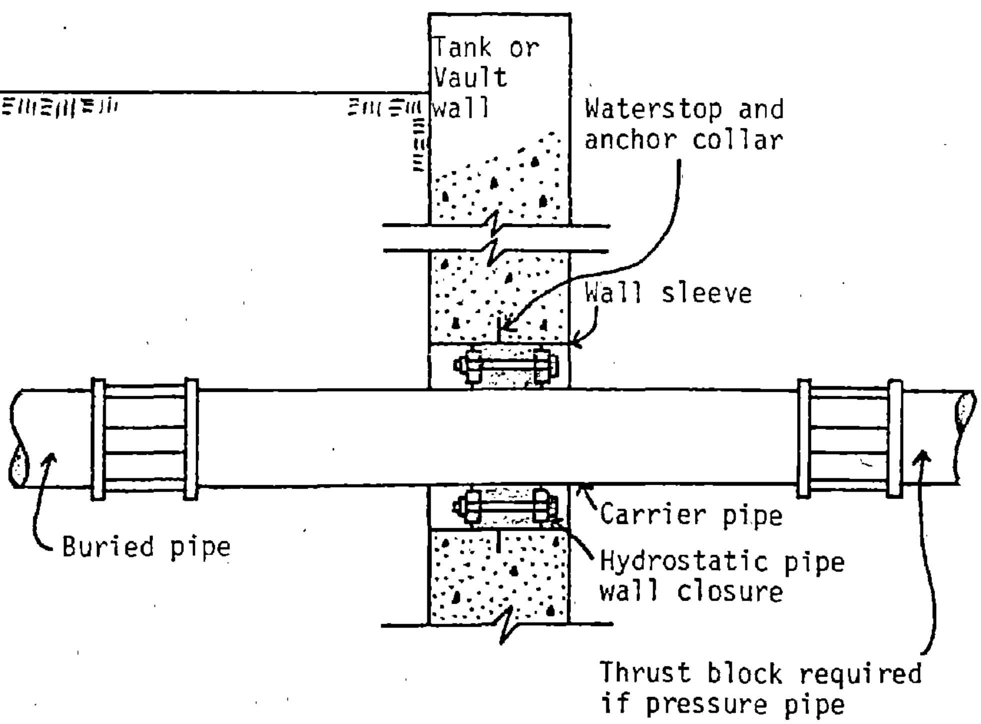


Figure VI-42. Flexible modular wall penetration seal (167).

valve should be located as close to the source as possible. The valve may be activated by pressure drop caused by pipe rupture, common for natural gas services, or it may be seismically activated. Seismically activated valves may be controlled by seismically activated switches which are currently available commercially. However, because seismically activated systems would shut off at a preset seismic acceleration, flow would be discontinued even if there was no damage to the piping system. The effect of possible interruption of flow should be considered, as it may interrupt process functions and thereby cause other problems. For example, disruption of a chlorine supply may allow undisinfected potable water to enter the distribution system. If a system is pressured by a pump, the seismically activated switch should shut off the pump as well as the valve to avoid possible pump damage. This is particularly critical for positive displacement pumps; when the discharge is valved off, the fluid pressure may increase beyond the system's bursting pressure, rupturing the piping or equipment.

BUILDING STRUCTURES

Seismic resistant design of building structures is beyond the scope of this report. The Uniform Building Code (UBC) (140) is a widely adopted building code addressing earthquake resistant design. The Tentative Provisions for the Development of Seismic Regulations for Buildings prepared by the Applied Technology Council (ATC 3-06) presents the "state of technology" in seismic resistant building design (141). ATC 3-06 is currently being reviewed. It is recommended that the design engineer refer to one of these publications or other local codes for seismic resistant building design criteria.

Before using an existing building code, the level of importance of the building must be established. Any building structure housing essential facilities should be designed as an essential facility itself. All water and sewage facilities discussed in this report are to be defined as essential facilities except those that may be out of service for up to 6 months (see discussion in Chapter III).

The Uniform Building Code includes an importance factor, I, as a multiplier in computation of lateral earthquake induced forces on buildings. This will account for the essential facility requirement.

ATC 3-06 defines "Seismic Hazard Exposure Groups." Essential facilities would be classified as being in Group III. ATC 3-06 then further assigns design groups based on the Seismic Hazard Exposure Group and the seismicity of the particular area. Specific criteria are included in ATC 3-06 for each design group.

The building and the enclosed facilities should be structurally compatible. A rigid building design would provide rigid support to any attached facilities. A flexible or ductile building may not adequately support a rigid attached system. Either the building and attached system should respond as a single unit, or flexibility should be designed into their attachment. Pipe wall penetrations exemplify this situation.

D. STORAGE TANKS

This section includes design considerations for both surface supported and elevated water storage tanks. Surface mounted tanks, while commonly constructed of steel, are sometimes built of reinforced concrete. Elevated tanks are usually supported on crossed braced steel structures. However, steel pedestal tanks are becoming more common. Reinforced concrete elevated tanks are common in some areas outside the United States.

Chapter VII of this report develops procedures to calculate the seismic forces to be considered in tank design. The American Water Works Association (AWWA) "Standard for Welded Steel Tanks for Water Storage" (D-100) (190) provides standards for the design of the steel tank structure. The purpose of this section is to highlight some of the "detailing" considerations for seismic resistant design.

GENERAL CONSIDERATIONS

Tanks should be located as close as possible to the area where water will be used, since the distribution system interconnecting the tank and system would then be less likely to fail. However, consideration should be given to a safe discharge route for the water should the tank fail.

Tanks should not be sited near faults. Areas with unstable soils such as hillsides, fill areas, and areas with a high liquefaction potential should also be avoided. Soil stabilization, although possible, is expensive.

FOUNDATIONS

If a tank is sited in an unstable soil area, the soil may be stabilized using vibroflotation or grouting. The tank could also be supported on piles.

Storage tanks should preferably be constructed on virgin soil and not on fill. Whenever a tank is constructed on fill, the fill material should be of uniform depth and compacted to a degree that will ensure comparatively high shear strengths and little future consolidation (045).

Particularly for surface mounted storage tanks, one method for improving seismic performance preventing shell wall buckling is to provide a concrete ringwall foundation. Figure VI-43 shows an example utilizing a ringwall structure. The concrete ringwall significantly strengthens the bearing capacity of the soil by limiting the lateral escape of loose soils. In addition to the ringwall, sand and gravel layers can be alternatively placed underneath the shell base plate and compacted. This limits differential settlement, and thus tank inclination and subsidence.

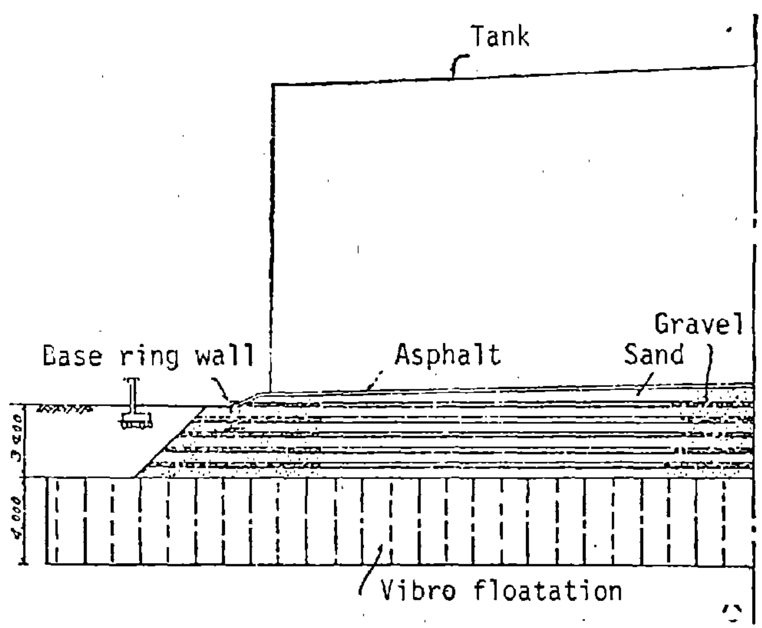


Figure VI-43. Method for strengthening the base for a surface mounted tank (098).

The design of elevated tank foundations should be adequate to transfer the loadings from the structure to the ground. Particular attention should be given to the overturning moment inducing an upward force on one side of the foundation. Some designers (191) have taken into account the seismic response of the foundation and surrounding soil mass in tank design.

The foundation design for crossed braced tanks can be limited to that resisting force required to cause the cross bracing to yield.

STRUCTURAL DESIGN

Surface Supported Tanks

Tanks should be structurally designed to resist the forces calculated using the procedures presented in Chapter VII. Maintaining the tank height to diameter ratios between 0.4 and 0.7 will control the seismic tank loadings.

A number of other design considerations are listed below.

- The tank must be designed to transfer the loading from the roof through the tank to the foundation.
- In order to strengthen roof-to-shell connections, abrupt changes in thickness between the shell wall and roof plate should be eliminated (Figure VI-44) (098).

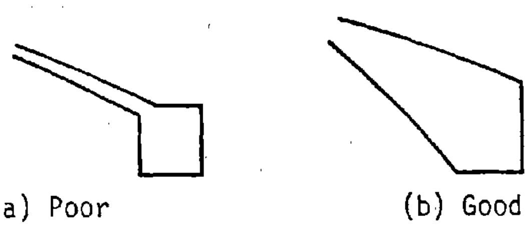


Figure VI-44. Roof-to-shell connections for storage tanks are improved by avoiding abrupt changes in plate thickness (a). Thickness changes should be gradual (b) (098).

- Roof buckling and broken roof-to-shell connections can be minimized by allowing enough freeboard to prevent the sloshing wave from coming into contact with the roof (094, 192).
- Roof rafters should be positively attached to the roof or braced laterally, not depending on friction between the roof and joist (192).
- The shell should be butt-welded from both sides to an annular bottom plate for unanchored tanks, resulting in a stronger joint to resist uplift (192).
- Radiographing of all vertical joints should be considered (192).
- Low hydrogen electrodes enhance the weld quality and toughness; these are particularly useful at the shell and shell-to-bottom joints.
- A36, A131-A,B,C and A516-60 steels are tough and weldable, and are recommended for tanks in seismic areas (192).
- To prevent uplift of the tank wall due to rocking, the bottom plate around the edge of the shell can be stiffened. By preventing uplift of the shell wall, the overturning moment that the tank develops can be reduced, thus reducing the amount of compression exerted on the shell wall opposite of uplift. Stiffening of the bottom plate can be accomplished by either increasing the thickness of the bottom plate or butt-welding an annular bottom plate to the shell wall. However, caution should be exercised in increasing the thickness of the bottom plate to avoid increasing the bending moment applied to the shell near the shell-to-bottom joint (094, 192).
- Shell buckling can also be reduced by increasing the thickness of the shell wall to accommodate compressive stresses (192).

AWWA D-100 (190) includes standards limiting the thickness and compressive forces in the bottom annular ring and bottom shell course. Beyond these limitations, the tank must be positively anchored to the foundation. In turn, the foundation must be designed to resist the uplift of the anchor bolts.

Figure VI-45 shows one possible detail for an anchor bolt assembly.

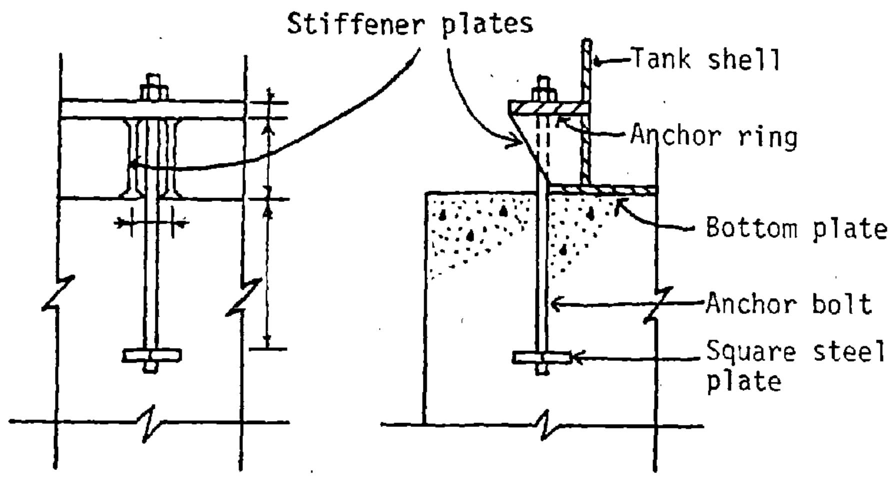
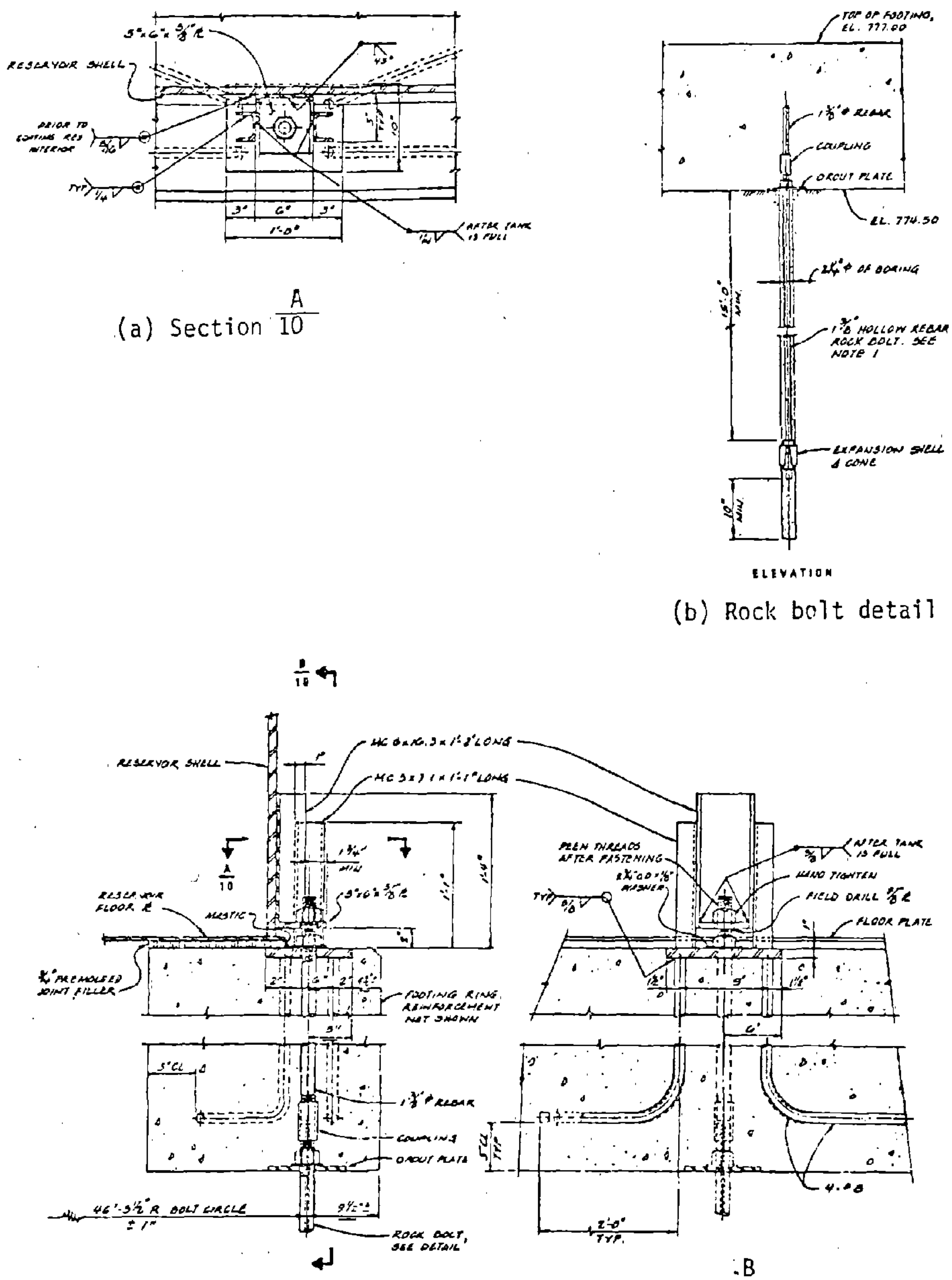


Figure VI-45. Details of anchor bolt design (192).

Figure VI-46 shows an anchor bolt assembly used by EBMUD that transfers the uplift forces to the rock below.

Anchor bolts should be designed so that for moderate earthquakes, they will anchor the tank without yielding. For major earthquakes, the bolts should be designed to yield before the attachment to the tank fails. This could allow the tank contents to be lost. The yielding anchor bolt will absorb some of the earthquake's energy.



(a) Section $\frac{A}{10}$

(b) Rock bolt detail

(c) Sectional elevation

(d) Section $\frac{B}{10}$

Figure VI-46. Tank hold down with rock bolt detail used by EBMUD (168).

Elevated Tanks

Cross braced elevated tanks should be designed so that yielding can develop in the bracing system before failure of a connection detail or buckling of a strut, when subjected to seismic loads (190). The yielding members will absorb energy, reducing the impact of the earthquake on the structure. Rods used for bracing should have upset threads (190). Energy could also be absorbed by using large deformable washers under retaining nuts (192).

ATTACHED PIPING

The AWWA standard D-100 (190) recommends that a minimum of 2 inches of flexibility in all directions should be provided for all piping attached to the shell or bottom of steel storage tanks. Flexibility of joints between storage tanks and attached piping is necessary to allow for differential displacements which might occur between the pipe and the tank.

Details of a possible flexible joint for a pipe connection to the shell wall are shown in Figure VI-47 (192). The restrained expansion type joint can be used to provide offset movement between the storage tank and the attached piping. This type of joint can provide a 4^o rotation from the straight position at each joint for pipelines up to 30 inches in diameter. The joint also permits some axial flexibility.

Other possible flexible connections include the use of ball joints, metal and rubber bellows and loops. Section C of this chapter should be referred to for greater detail.

For bottom piping connections for unanchored flat bottom tanks, AWWA D-100 recommends that the connection should be located inside the shell far enough to prevent breakage of the connection due to uplift of the tank, at least 12" inside the unanchored bottom hold down as calculated.

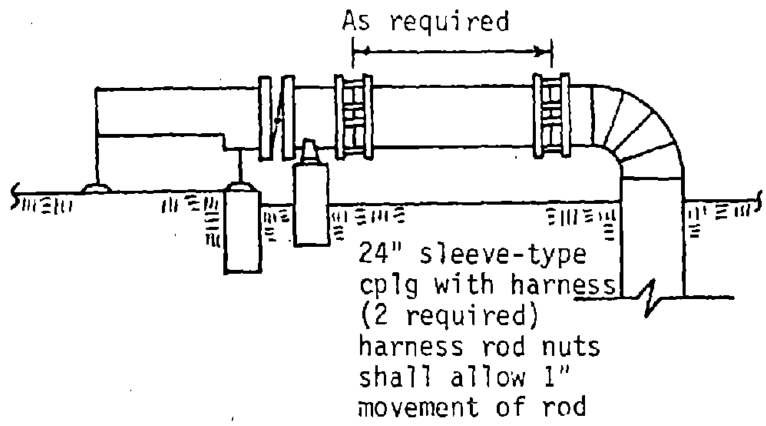


Figure VI-47. Schematics of a flexible restrained joint for pipe connections to storage tanks (192).

Consideration may be given to automatic tank shut-offs that will activate when subjected to earthquake motions or when large flows are encountered (from downstream pipe failure). This may prevent the loss of water. On the other hand, it may be inappropriately activated when the water is needed for firefighting.

E. COSTS OF SEISMIC RESISTANT CONSTRUCTION

Seismic resistant construction costs will generally equal or exceed those not taking seismic resistance into account. The additional cost would be dependent on the particular situation. In many instances, preliminary planning with seismic resistance in mind can avoid major seismic resistant construction costs. A detailed, cost-effective analysis is site specific and, therefore, is beyond the scope of this report. This section will, however, discuss various cost aspects of earthquake resistance.

Preliminary planning for site selection should select areas not subject to soil failures. If there are no stable sites available, the soil can be stabilized, the facility set on piles, or, because of high costs, the instability disregarded.

Pipeline design for the most part will not change. Ductile iron and PVC pipe, both with push-on joints, now in common use, are recommended. Flexible connections will add a nominal cost to construction. In highly seismic areas with unstable ground, restrained joint pipe may be employed, adding a moderate amount to the construction cost.

Concrete tank walls will cost less than 1 percent more when designed to resist seismic forces in the highest seismic risk zone. The increase is nominal due to the allowance for natural allowable stress increases in seismic design.

Equipment anchorages, to the level required to resist overturning, would cost approximately the same as a non-seismic anchorage system. The only increase would be in the cost of the larger diameter bolts. The major cost, labor, would remain the same. Resilient anchorage systems must be provided with stops, thus increasing their cost.

Many vulnerable equipment designs can be avoided in the selection process if the design team believes they will not withstand earthquake loadings. Alternate equipment can often be selected at little or no additional cost.

Equipment qualificaiton will initially be expensive. However, if seismic qualification and testing are adopted industry-wide, economy of scale will take over. The structural changes required in most pieces of equipment to withstand seismic testing will be nominal.

Plant piping lateral bracing will be required in most instances. If the bracing is installed at the same time as the pipe, the increased costs would be nominal.

Flexible connections are currently commonly used between pumps and piping to reduce vibrations and add to the ease of installation. This will not change. Throughout the plant, some additional flexible connections will be added. In severe seismic regions, ball-joints may be employed. Some additional cost will be involved.

The selection of the building type will significantly effect its resistance to earthquakes. Construction increases would be nominal, if any, for low profile buildings commonly found at treatment facilities.

Miles (192) presented a graph showing the increased cost for seismic resistant construction of surface mounted steel storage tanks. The information was based on cost data from seven reservoir projects. Cost increases were a result of thicker shells and bottom plates, extra freeboard, flexible piping connections, and testing. The results are shown in Figure VI-48.

Seismic resistant design considerations should be incorporated into a project from the beginning. Additional costs could range from zero to substantial, depending on the situation. However, retrofitting a facility to resist earthquakes once it has been constructed, or replacing or repairing earthquake damaged systems, will be much more expensive.

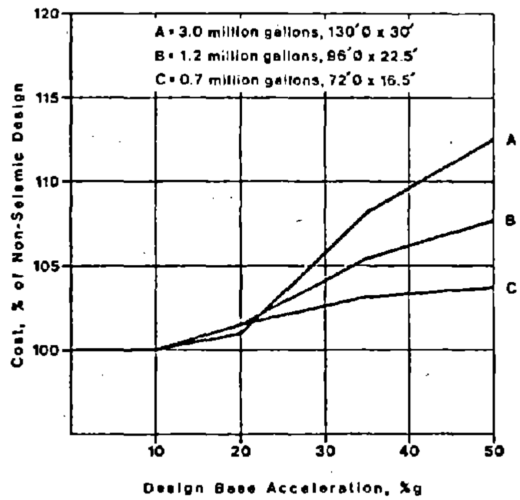


Figure VI-48. Cost increase vs. base acceleration (192).

CHAPTER VII

CRITERIA FOR DETERMINATION OF SEISMIC INDUCED LOADINGS ON WATER SUPPLY AND SEWAGE FACILITIES

There are currently no widely accepted codes defining criteria to calculate seismic induced loadings specifically for water and sewage facilities. This chapter presents a suggested code to fill that void. The suggested code is based on the Tentative Provisions for the Development of Seismic Regulations for Buildings recently compiled by the Applied Technology Council (ATC 3-06) (141), associated with the Structural Engineers Association of California. ATC 3-06 was developed for building design; these provisions have been tailored to be used for the design of water and sewage facilities. Several major additions have been made from other sources, including tank and retaining wall design. It is recommended that a copy of ATC 3-06 be obtained by the user of this report, for it presents the development of the basic approach taken by both ATC 3-06 and these criteria. Where applicable, comparisons are made between the suggested code and the Uniform Building Code (UBC) (140), prepared by the International Conference of Building Officials, as UBC, or modifications thereof, includes the most widely accepted general seismic resistant design criteria in the United States.

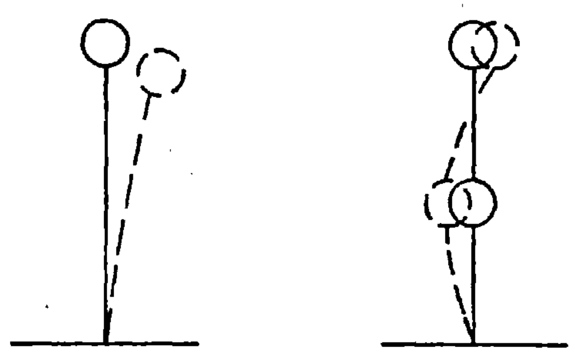
The criteria contained in the suggested code are comprised of specific procedures for calculating seismic induced loadings on rigid and resiliently mounted equipment and piping, buried and surface mounted tanks and vaults, elevated tanks and structures, elements immersed in water, and tank and retaining walls. Criteria for combining seismic induced loadings with other standard loadings, i.e., dead, live and snow loadings, are also included.

This suggested code is intended to be used as a starting point for discussions to develop a widely accepted code. It is therefore presented in a format that includes discussion of the development of the criteria, rather than in a conventional concise code format. The suggested code is indented and delineated with a solid line in the margin to separate it from the discussion which follows. Equations, tables and figures included in the suggested code are numbered VII-1, 2...etc. Equations, tables and figures that are included in the discussion but not in the code are numbered VII D-1, 2...etc.

A. GENERAL DESIGN APPROACH

Structures including equipment and piping are designed to respond elastically to earthquake accelerations provided that these accelerations do not exceed the level chosen or specified for design. Elastic deformation of the structural material occurs but, by definition, the material returns to its initial position once the force induced by the acceleration is removed. These criteria are developed on the basis of linear elastic deformation, where the ratio of the displacement of a structure to the force applied to the structure, the strain, essentially is linear. They limit the forces induced on the structural material to a level less than or equal to the member's strength (limiting energy absorbing [damping] capabilities of the structural system). The limitation of motions by system damping (comparable to a shock absorber) requires that the system be designed to withstand the full acceleration transferred to it from the ground.

Simple structural systems are generally encountered in water and sewage facilities. A typical system can be represented as an elastic single degree of freedom system, as depicted in Figure VII-D-1.



a. One degree of freedom   b. Two degrees of freedom

FIGURE VII D-1. Degrees of Freedom

An elastic single degree of freedom system will respond to each earthquake in a characteristic manner depending on the system's natural frequency and damping. Damping is expressed as a percentage of critical damping, where critical damping is the minimum energy absorbing capability of a structure that will prevent the structural system from oscillating when a velocity or displacement is imparted, but will allow the structure to return to its undeformed shape after an infinitely long time (001).

For a given percentage of damping, the maximum response to any given earthquake can be plotted using the periods of elastic single degree of freedom oscillation (the inverse of its natural frequency) as an abscissa and the acceleration, pseudo velocity and/or deflection as an ordinate. This is commonly referred to as the response spectra. For design spectra based on a specific earthquake, smooth curves are usually drawn, as true response spectra are very irregular. It is highly unlikely that a specific earthquake motion will be repeated in the future, thus making the applicability of actual response spectra limited for design. For a Code design spectra, many response spectra are reviewed, and one is selected that produces accelerations that vary with natural frequency in low frequencies but have a constant acceleration for high frequency systems.

The "Typical Design" given in Section C of this chapter assumes that the natural frequency of the structures considered is greater than 2.5 cycles per second. The frequency response range from approximately 2.5 to 9 cycles per second is subjected to the highest accelerations within the spectra of frequencies associated with earthquakes. Structures with lower natural frequencies can be conservatively designed using this approach as well.

For any given earthquake, by increasing a structure's natural frequency from 5 to 20 cycles per second (i.e., by changing its structural characteristics), the response acceleration may be reduced by as much as 60 percent. Increasing the structure's natural frequency to 33 cycles per second may reduce the response acceleration by as much as 80 percent. Design engineers in the nuclear industry attempt to keep the natural frequency of structures above 33 cycles per second for this reason. In order to simplify design, the recommended criteria contained herein do not allow a reduction in acceleration by increasing the structure's natural frequency. The earthquake acceleration of a structure decreases quickly as the structure's natural frequency decreases below 2.5 cycles per second (193) ; however, high velocities and displacements of the structures then may become a problem in this low frequency range. A frequency dependent reduction of the design acceleration is permitted for low natural frequency structures; tanks and towers are typical of such structures.

Two alternative design approaches are presented in this chapter: strength design and working stress design. These are based on different basic design philosophies; the choice of approach should be based on the material specifications with which they are being used.

The strength design method relates the limit of usable strength of an element to a factored load effect on the member. This contains a built-in factor of safety in the determination of the usable strength and in the factor used in the determination of the load effect. The usable strength can be a yield strength, fracture strength or buckling strength design, whichever governs. The basic strength design format has been used in concrete design [ACI 318 (164)] for many years and is currently being considered as an alternative design procedure by AISC and AISI.

Working stress design nominally envisions straight line stress - strain relationships on member cross-sections. Levels are set such that they include the 1/3 increase usually given in codes and specifications for load combinations involving earthquake stress. The working stress design format is currently used in AISC and AISI specifications.

B. GENERAL EQUATION DEVELOPMENT

In this section, the general equation will be developed that will define the forces induced on water and sewage system structures from seismic accelerations. The equation will include terms for both rigid (short period) and flexible (long period) structures. A simplified definition will be presented for equation variables with reference to detailed discussions of each later in the chapter. The equations in each subsection will then be developed from the general equation.

The design approach taken in patterned after ATC 3-06. Many water and sewer related structures are rigid. In accordance with the design approach taken by Chapter 8 of ATC 3-06, the magnitude of the response of these structures will not be a function of their fundamental period. Long period systems do, however, have a period related response. The period of motion of an elevated tank structure and the period of the sloshing water contained in a surface mounted tank fall into this category.

The overall approach taken in developing the following design criteria, including period related responses, is discussed in detail in ATC 3-06, Chapter 4, "Equivalent Lateral Force Procedure," and Chapter 5, "Modal Analysis Procedure." Definitions of terms included in the formulation below are the same as those included later in this chapter.

The basic ATC 3-06 equation is:

$$F_s = V = C_s^{\star} W \qquad \text{(VII D-1)}$$

where:

F_s = seismic induced force

V = base shear (ATC 3-06)

$C_s^{\star}$ = ATC seismic design coefficient representing a modified seismic acceleration coefficient (not related to C_s in these criteria) where:

$$C_s^* = \frac{1.2\ A_v\ S}{RT^{2/3}} \qquad \text{(VII D-2)}$$

but, for rigid structures, need not be greater than

$$\frac{2.5\ A_a}{R} \qquad \text{(VII D-3)}$$

where:

A_a = seismic coefficient representing seismic accelerations used when analyzing short period structures (Section C)

A_v = seismic coefficient representing seismic accelerations used when analyzing longer period structures (Section D)

S = soil factor varying from 1 to 1.5 depending on the local soil conditions (Section D)

R = response modification factor depending on the particular structure seismic response characteristics (Section D)

T = period of the structure (Section D)

One model used to simulate the response of surface mounted tanks by Housner (095) includes both the period related and non-period related terms for C_s^*. Other structural systems generally include either the period related or the non-period related term.

The combined equation takes the form:

$$F_s = \left[\frac{2.5\ A_a}{R}\ (W_a) + \frac{1.2\ A_v S}{RT^{2/3}}\ (W_b)\right] \qquad \text{(VII D-4)}$$

where:

W_a = weight of the structure responding directly with the ground motion (rigid response)

W_b = weight of the structure with a period related response, e.g., either water or weight of a tower

A seismic coefficient, C_s, is introduced to provide a method to define varying levels of seismic resistance various components are required to have (Section C). To relate the C_s used in these criteria to the overall ATC 3-06 approach, let C_s equal 2 (strength design) (see Table VII-1), which approximates a moderately critical situation.

Multiplying the right portion of equation VII D-4 by the factor $C_s/2$ (which equals 1) yields the following equation:

$$F_s = C_s \left(\frac{1.25\ A_a}{R}\ W_a + \frac{0.6\ A_v S}{RT^{2/3}}\ W_b \right) \qquad \text{(VII D-5)}$$

which, upon rounding, yields:

$$F_s = \frac{1.2\ C_s}{R} \left(A_a\ W_a + \frac{A_v S}{2\ T^{2/3}}\ W_b \right) \qquad \text{(VII D-6)}$$

Let $C_1 = \frac{A_v S}{T^{2/3}}$ which from equation VII D-2 is $.83\ R\ C_s^*$ (VII D-7)

Similar to C_s^* in Chapters 4 and 5 of ATC 3-06, C_1 need not be taken as greater than the values given by the following formulae:

$C_1 = 2.0\ A_a$ (rigid response, from equation VII D-3) (VII D-8)

$C_1 = \frac{2.5\ A_v S}{T^{4/3}}$ (long periods, from equation V-3b from ATC 3-06) (VII D-9)

Combining equations VII D-6 and VII D-7:

$$F_s = \frac{1.2\ C_s}{R} \left(A_a\ W_a + \frac{C_1}{2}\ W_b \right) \qquad \text{(VII D-10)}$$

Equation VII D-10 is the general equation from which the basic equations in the following subsections can be derived. The basic equations can be developed by substituting terms and values appropriate to the type of structure being analyzed for weights, W_a and W_b, the response modification factor, R, and multiplying the results by modification factors to refine the results. The basic equations are developed as follows:

Equation VII-2 (page VII-11): Typical Design - Rigid Non-period Related Response

Let $W_a = W$,

let $W_b = 0$,

include an amplification factor a_x,

let R = 2

Equations VII-7, 8 (page VII-27): Horizontal Forces on Surface Mounted Tanks, both non-period and period related

Let $W_a = W_t + W_r + W_1^{CT}$ (or W_1^{RT}),

let $W_b = W_2^{CT}$ (or W_2^{RT})

Equation VII-17 (page VII-42): Horizontal Forces in Elevated Tanks, period related

Let $W_a = 0$,

let $W_b = W_r$

C. TYPICAL DESIGN, HORIZONTAL FORCES

This section presents suggested techniques and criteria for calculating horizontal earthquake induced forces on rigidly mounted structures, equipment and piping, examples of which include pumps mounted on concrete with anchor bolts, and pipes suspended with rigid pipe hangers.

STRENGTH DESIGN

Structures, piping and equipment should be designed to resist the effects of the seismic force, F_s, applied to the center of gravity of the structure's mass, where:

$$\phi R_n \geq F_s \qquad \text{(VII-1)}$$

ϕ= strength reduction factor to represent calculated resistance as a mean resistance

R_n= calculated nominal strength

F_s= minimum design seismic induced force for use with the strength design method, determined as follows:

$$F_s = 0.6 \; A_a \; C_s \; a_x \; W \qquad \text{(VII-2)}$$

where:

A_a = seismic coefficient representing the Effective Peak Acceleration as shown on Figures VII-1 and VII-2, as a percent of gravity

C_s = seismic coefficients for components of the system as shown in Table VII-1.

TABLE VII-1. Seismic Coefficient for Strength Design

C_s	Equipment Type
1	Long term makeshift operation or shutdown is acceptable up to 6 months.
2	Intermediate shutdown is acceptable up to 2 weeks.
3	Continuous operation is essential.

a_x = amplification factor taking into account the building response for equipment mounted in or on the building

$$= 1 + \frac{(h_x)}{(h_n)} \qquad \text{(VII-3)}$$

where:

h_x = height of the floor on which the equipment is mounted

h_n = total height of the building

W = effective weight of equipment of component and contents

WORKING STRESS DESIGN

Structures, piping and equipment should be designed so that the allowable working stresses will not be exceeded when a force is applied to the center of gravity of the equipment mass, where:

$$\frac{\phi}{\gamma_m} R_n = R_w \geq F_w \qquad \text{(VII-4)}$$

γ_m = mean load factor [average of different load cases (i.e., live, dead, earthquake) where the load factor represents the uncertainty of specifying the load by giving the mean (e.g., specifying 100 psf when the actual loading may vary from 85 to 115 psf)]

R_w = calculated working stress capacity

$$F_w = 0.6\ A_a\ C_w\ a_x\ W \qquad \text{(VII-5)}$$

and:

ϕ, R_n, A_a, a_x, and W are the same as those used in the strength design method

C_w = seismic coefficients for components of the system as shown in Table VII-2

TABLE VII-2. Seismic Coefficient for Working Stress Design

C_w	Equipment Type
0.8	Long term makeshift operation or shutdown is acceptable up to 6 months.
1.6	Intermediate shutdown is acceptable up to 2 weeks.
2.4	Continuous operation is essential.

Effective Peak Acceleration (EPA) and Effective Peak Velocity (EPV), introduced later in Section B, "should be considered as normalizing factors for construction of smoothed elastic response spectra....The EPA is proportional to spectral ordinates for periods in the range of 0.1 to 0.5 seconds" (2 to 10 cycles per second), "while the EPV is proportional to spectral ordinates at a period of about one second" (1 cycle per second) (141). The EPA should therefore be used for calculation of earthquake induced forces on high frequency structures such as rigid, rigidly mounted equipment, and the EPV should be used to calculate the earthquake induced forces on low natural frequency structures such as tanks.

The EPA and EPV are not the actual accelerations encountered but take into account the level of damping, a proportionality constant, earthquake duration, frequency, and instantaneous peak vs. maximum continuing accelerations. For a detailed discussion, refer to ATC 3-06.

EPA and EPV are replaced, respectively, by the dimensionless coefficients A_a and A_v, which are decimal fractions calculated by dividing the acceleration by gravity. Maps showing the areas of varying values of A_a and A_v are presented in Figures VII-1 through VII-4. These maps are included in ATC 3-06, representing areas with an S of 1.0 (firm ground); S values are described later. The ATC 3-06 map included in this report is a contour map. This form was included for ease of reproduction. Included in ATC 3-06 is a larger scale, multi-colored map delineating coefficients by county line. That map may be used as a reference if greater detail is required.

These ATC maps given in Figures VII-1 through VII-4 were used because they represent the most recent national seismic zone analysis for Code purposes. While they represent the "state of technology" in delineating seismic zones, it must be recognized that earthquake zonation is probabilistic engineering with no guarantee of its effective correctness. In addition, the authors of this report had no part in developing these maps and are therefore not responsible for their contents.

The map user should keep in mind that the determination of seismic map areas is based on both earthquake magnitude as well as its probability of occurrence. Consideration may be given to increasing the design acceleration parameters in areas with a history of significant earthquakes with low occurrence probabilities.

The A_a and A_v values shown on these maps are regional in nature and do not take into account microzonation, which is local seismic zoning accounting for local tectonic conditions. It may be advisable to perform a detailed local seismic study in highly seismic areas. This may be particularly useful in the western United States where earthquake foci are typically shallow and proximity to the epicenter may thus substantially effect the earthquake motion intensity at the facility site.

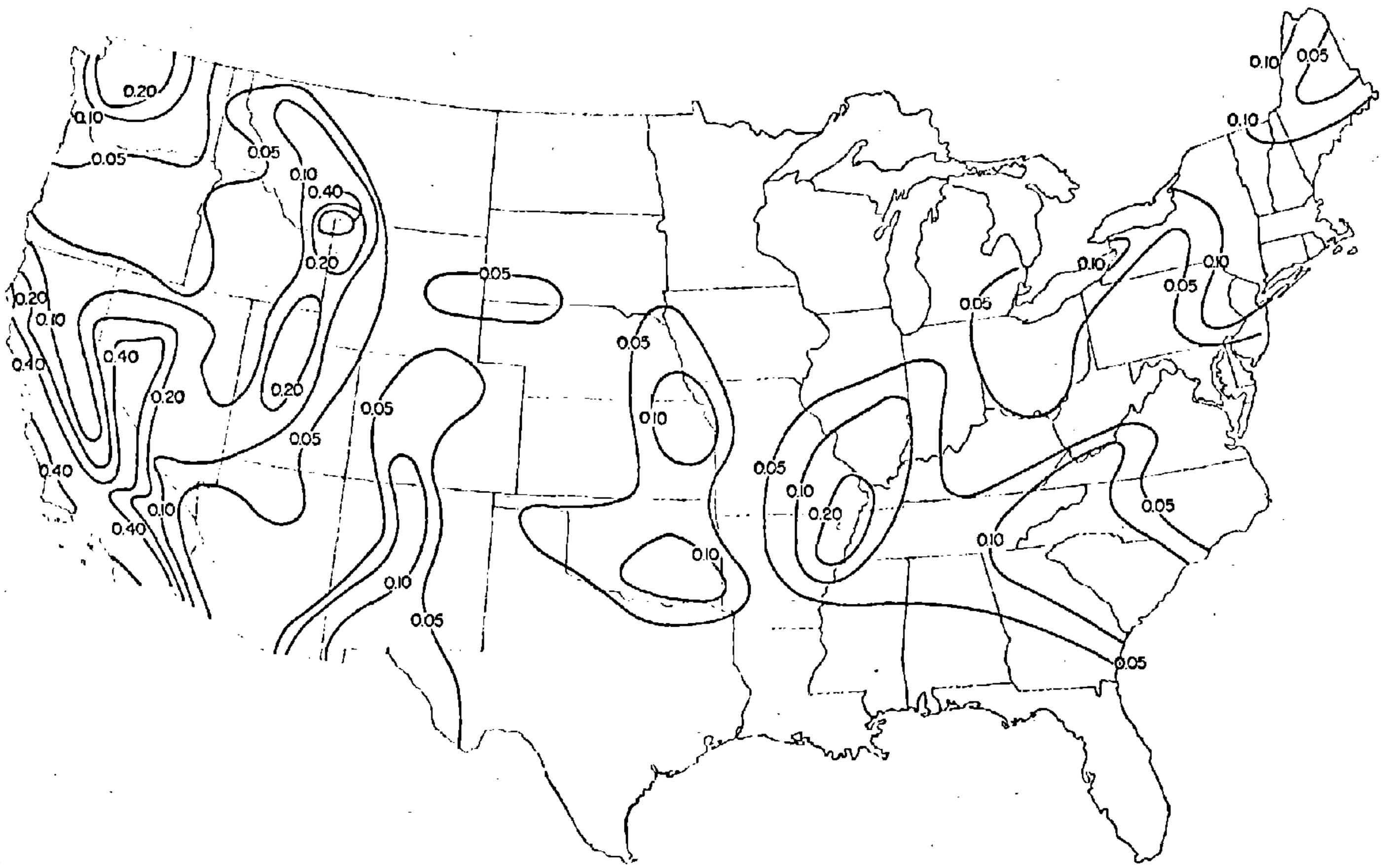


Figure VII-1. Contour map for seismic coefficient A_a (141).

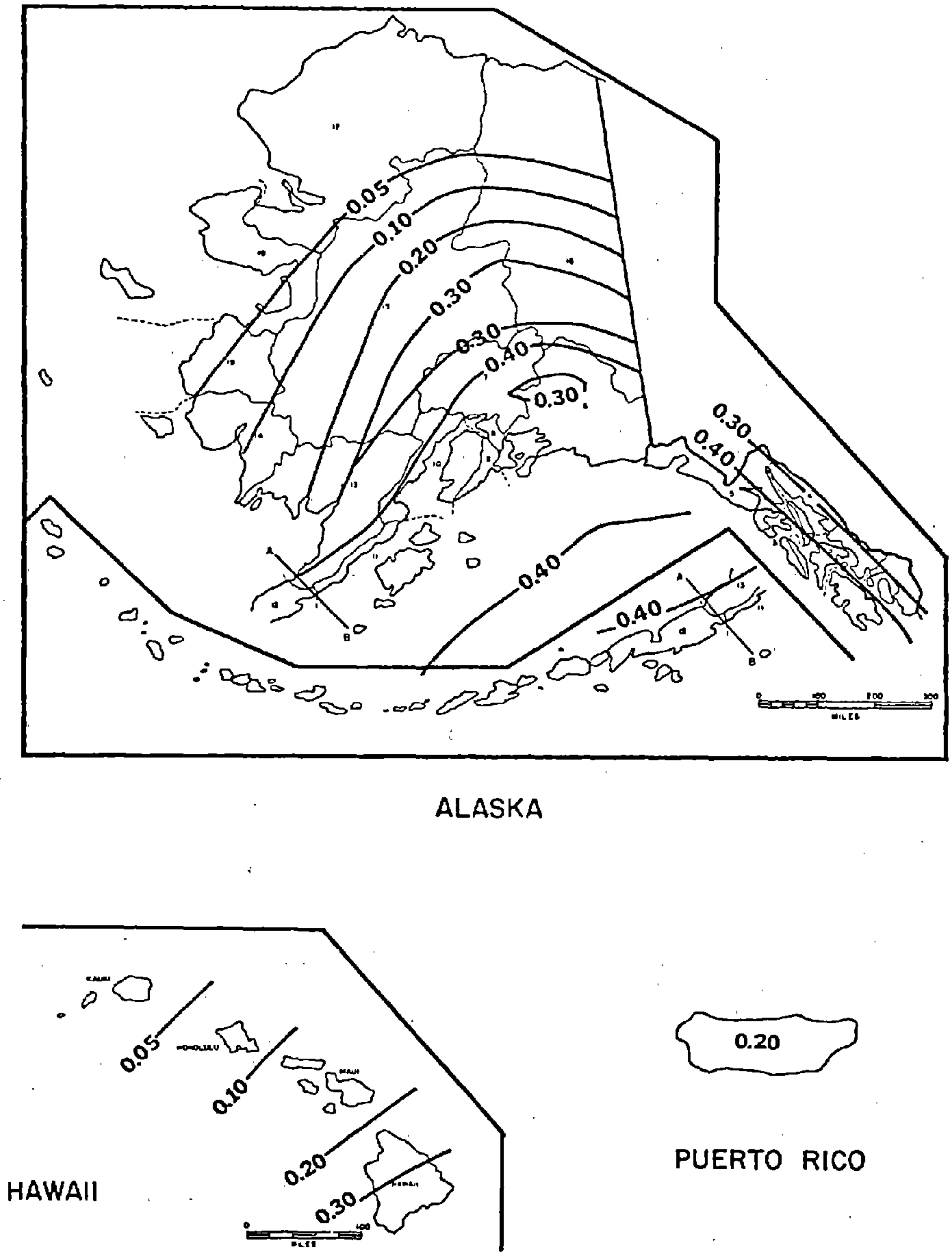


Figure VII-2. Contour map for seismic coefficient A_a (141).

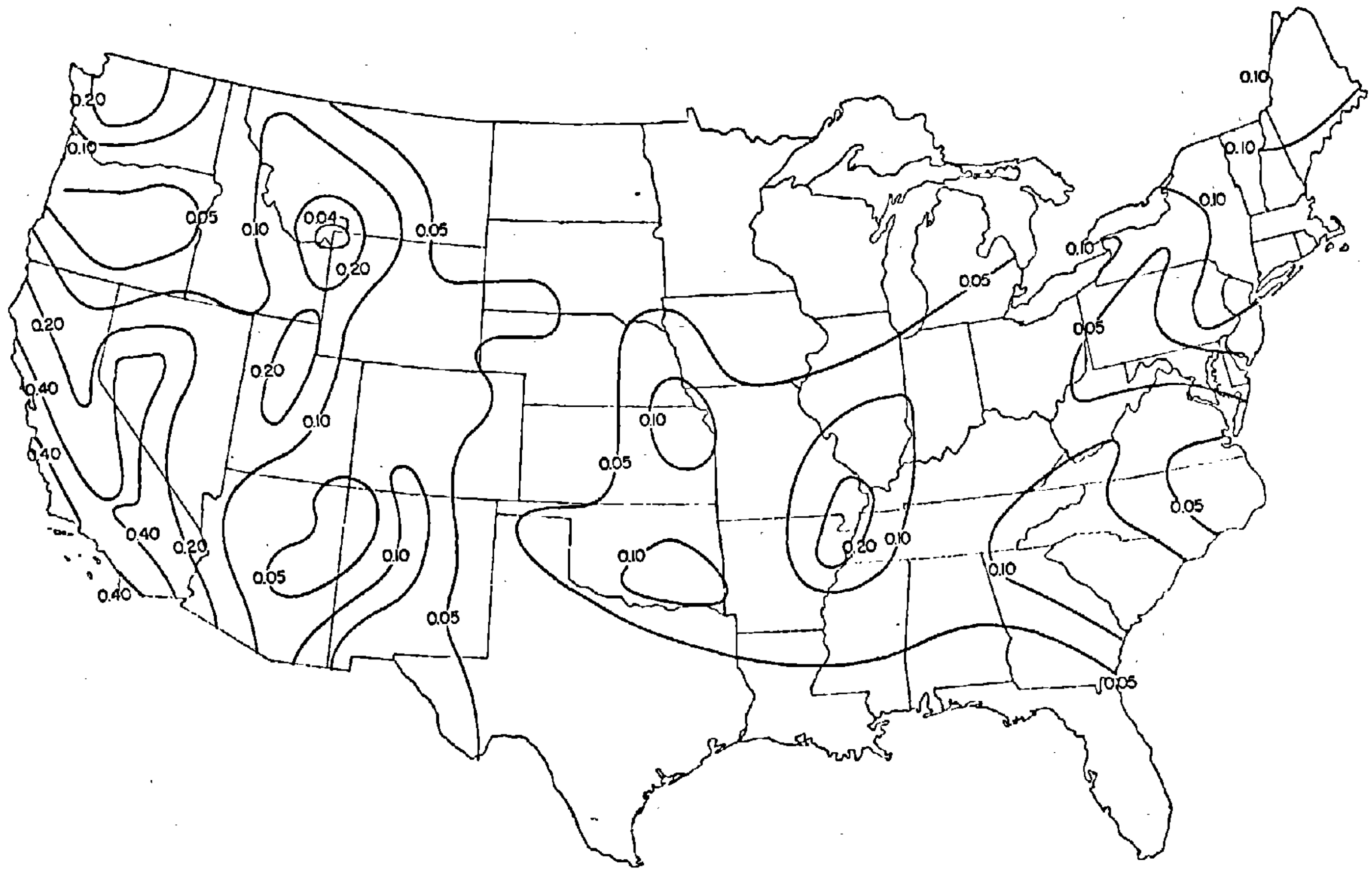


Figure VII-3. Contour map for seismic coefficient A_v (141).

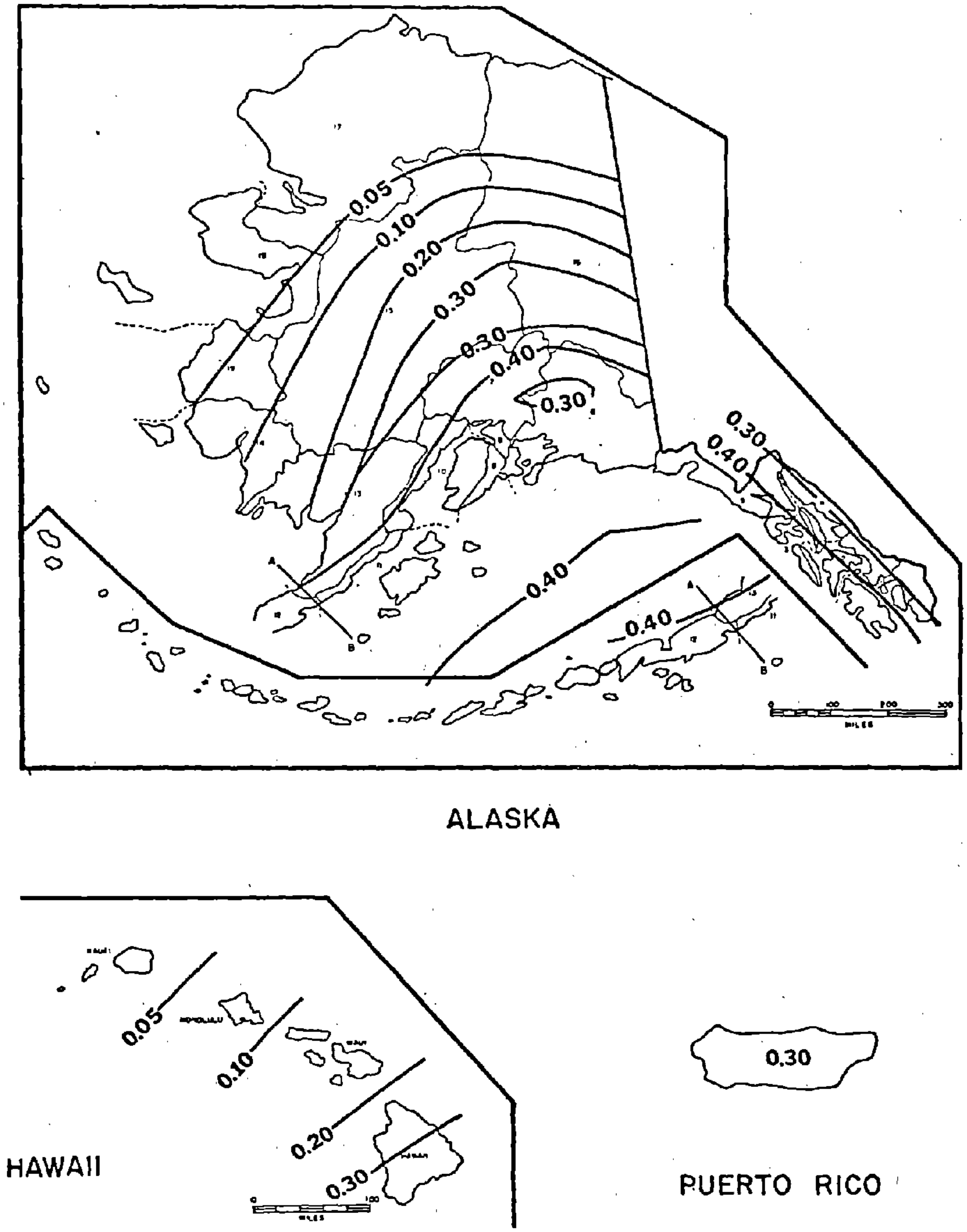


Figure VII-4. Contour map for seismic coefficient A_v (141).

The maps show areas in which the A_a and A_v would have a 10 percent probability of being exceeded in 50 years. An increase in the design acceleration of 20% would decrease this probability of occurrence to approximately 5%, and doubling the design acceleration would reduce this probability to approximately 0.5% (141). This along with other factors is considered in establishing the seismic coefficient values. A detailed discussion of these maps is included in ATC 3-06.

The higher the acceleration value A_a or A_v chosen, the more conservative the design. In many instances, a higher value may be used that will only slightly increase the cost of the structure. The highest economically reasonable value of acceleration should be used, due to the unpredictability of earthquakes.

The seismic coefficients C_s and C_w used in these criteria are modifications of those presented in ATC 3-06; comparisons are made with ATC and UBC below. C_s replaces both the performance criterion P and the seismic coefficient C_c found in Chapter 8 of ATC 3-06. For comparing maximum values, a P value of 1.5 is selected, as all water and sewer system facilities necessary to maintain the primary function of the system should have a "superior" ATC overall performance rating because of the "lifeline" nature of the systems. Multiplying this P value by a C_c value of 1.2 (a median value of those listed in ATC 3-06) yields a product of 1.8. This is the same value obtained by multiplying a C_s value of 3.0 (maximum in these criteria) by a factor of 0.6. The 0.6 factor, derived previously, is equal to $\frac{C_c}{R} = \frac{1.2}{2} = 0.60$

The UBC, in some cases, uses working stress load values. A value of $C_p IZ$ (UBC horizontal force factor, occupancy importance factor, seismic risk zone numerical coefficient, respectively) equals (0.3) (1.5) (1), or 0.45. This can be compared to maximum values in these recommended criteria also using working stress load values. Using a C_w of 2.4 and an A_a of 0.4, the comparable value would be (0.6) (2.4) (0.4), or 0.58, which is 28% higher than the UBC value. It should be noted that seismic risk zone numerical coefficients must be included when comparing UBC and the suggested criteria as they are set at different levels.

The minimum effective seismic coefficient C_s in these criteria is 1, which, when multiplied by 0.6, yields a product of 0.6. The minimum ATC 3-06 value of C_c is 0.67 (equipment that has no effect on loss of life or secondary damage such as fire). To obtain a value of 0.6, the ATC 3-06 performance rating P would be equal to 0.9, which lies between the "good" and "low" ATC performance rating. The actual minimum P value used by ATC 3-06 is 0.5, which, when multiplied by the minimum C_c value of 0.67, yields 0.33, as opposed to 0.6 calculated using these criteria. Even though this ATC value is comparatively low, it is assumed that the structure might not fail because of non-seismic design strength.

The maps showing the values of A_a and A_v (Figures VII-1 through VII-4) are based on a seismic coefficient for strength design, C_s, of 2 (working stress design, C_w, 1.6). As the seismic coefficient C_s used for designing structures increases above 2, the modified values of A_a and A_v map contours (i.e., A_a [C_s]) would increase, indicating that the structure is being designed to withstand seismic induced motions that are stronger but occur less frequently than presently indicated on the maps. Increasing the seismic coefficient would decrease the probability of an earthquake acceleration occurring during the lifetime of the structure that would exceed the design acceleration. An increased seismic coefficient is also included to account for complex amplifications and responses not otherwise considered.

The maximum C_s value of 3.0 is to be applied to system components that are essential in meeting the operating goals of the system immediately following an earthquake. A water system should be capable of supplying disinfected water in quantities sufficient to fight fires. Thus, if a treatment plant is the sole source of water for a system, the components of the plant required to provide continuous hydraulic flow, i.e., pumps, channels, etc., and disinfection would be assigned a C_s value of 3.0, with other components of the facility assigned comparable values as discussed in Chapter III. If, however, an alternate water source could provide an adequate supply for up to two weeks, each supply facility's critical components would be assigned a reduced C_s value of 2.0. Pump stations and finished water storage tank design can be considered in a similar manner. If a particular facility is required to meet fireflow demand following an earthquake, it should also be assigned a C_s of 3 for design purposes.

Sewer system facilities required to provide hydraulic flow and disinfection following an earthquake are in the same category. This requirement may be somewhat easier to attain in sewage systems, as many rely on gravity flow.

Equipment piping or structures critical in times of disaster or whose damage may cause loss of life of operating personnel, or may cause fires to critical facilities, also require a C_s rating of 3. These include fire and smoke detection and suppression systems, boilers, incinerators, water heaters and toxic chemical storage and supply systems, such as chlorination systems.

Equipment and facilities designed specifically for emergency response, such as emergency power supplies and communication equipment, are to be considered critical components and should be assigned a C_s of 3.

Components of a system above, adjacent to, supporting, or otherwise interacting with a structure assigned a C_s of 3 must also be assigned a C_s of 3, as their failure may cause the critical component to function unsatisfactorily.

The relationship among components within a treatment system are discussed in detail in Chapter III. However, the relationship between the treatment facility and other supplies, storage tanks, and transmission and distribution lines must be evaluated on a system-by-system basis. Fire flow demands can be obtained from insurance rating organizations. The engineer should assign a C_s value ranging from 1 to 3 to all system components.

An analysis should be made to determine the economic impact of an increase in C_s factor. Frequently a major increase in C_s has little affect on costs, so a cost-effective increase can be made in the design force that will minimize the eventual earthquake hazard.

The amplification factor, a_x, is taken directly from ATC 3-06 and accounts for the response of the building in or on which system components are mounted. The factor is included in ATC 3-06 in part to motivate the design engineer to locate major equipment as low in the building as possible.

The effective weight, W, is that weight which contributes to the force induced on the structure by the earthquake acceleration. In most cases this would include the weight of the equipment, the portion of the structure supporting the equipment and responding with it (if the structure is flexible) or the portion of the structure above the pivot point in overturning, and the contents, liquid or dry. If the height of a cylinder containing liquid is less than 0.75 times the diameter, a secondary effective weight induced by sloshing of the liquid may be taken into account in design if found to be significant. If the engineer considers this secondary effective weight to be substantial, he may calculate its effective weight and response using the procedure presented in Section D of this chapter, Surface Mounted or In-Ground Tanks. The added weight of water responding with an immersed structure, included in Section E, is also considered effective weight.

The seismic induced force calculated for strength design, F_S, is 1.25 times that for working stress design, F_W. This factor is represented by the use of the appropriate seismic coefficient, C_S or C_W, respectively. The theoretical difference between the two approaches is that the probable maximum loadings are actually represented in strength design, whereas a working load, which is somewhat less than the probable maximum load, is represented in working stress design. In working stress design, the difference between working and maximum loading conditions is accounted for by lowering the material's design strength. The two methods are presented as a convenience to the design engineer so that the method used will be compatible with material specifications and codes used. The end result should, however, be approximately the same. Figure VII D-2 illustrates the two approaches.

COMPARISON OF STRENGTH AND WORKING STRESS METHODS

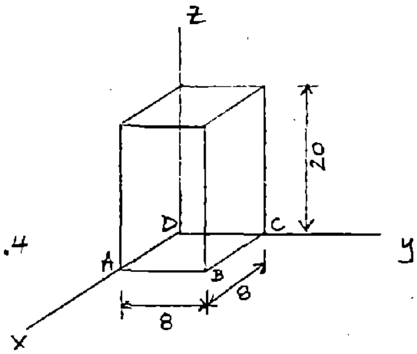
ASSUME:

OBJECT WITH DIMENSIONS SHOWN, AND WEIGHT (W) OF 100, WITH CENTER OF MASS AT CENTER OF VOLUME.

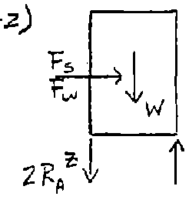
OBJECT BOLTED TO FOUNDATION AT EACH OF FOUR CORNERS.

FROM MAP, $A_a = 0.40$

CONTINUOUS OPERATION ESSENTIAL, $C_S = 3$
$C_W = 2.4$



COMPARE COMPUTATIONS OF UPLIFT AT CORNER 'A' USING STRENGTH AND WORKING STRESS METHODS.



R_A^z = REACTION AT CORNER A

STRENGTH

$F_S = 0.6 A_a C_S a_x W$ (EQ. VII-2)
$= 0.6 (0.40)(3)(1)(100)$
$= 72$

LET $Q = R_A^z$

EARTHQUAKE EFFECT:

$Q_x = Q_y = \frac{1}{2} \frac{F_S (10)}{8} = 45$

$Q_z = \frac{1}{4} (.6) F_S = \frac{1}{4}(.6)(72) = 10.8$

$Q_{E\ MAX} = \sqrt{Q_x^2 + Q_y^2 + Q_z^2}$ (EQ. VII-23)
$= \sqrt{(45)^2 + (45)^2 + (10.8)^2} = 64.5$

DEAD LOAD EFFECT:

$Q_D = \frac{1}{4} W = 25$

$Q = 0.95 Q_D - Q_{E\ MAX}$ (EQ. VII-25)
$= 0.95(25) - 64.5$
$= -40.8$

ASSUME SPECIFIED WORKING STRESS VALUE OF BOLT = 25

PERMISSIBLE STRENGTH OF BOLT
(1.7 TIMES BASIC WORKING STRESS, AISC SEC. 2.8 (540))
$= 1.7 (25)$
$= 42.5 > 40.8$

WORKING STRESS

$F_W = 0.6 A_a C_W a_x W$ (EQ. VII-5)
$= 0.6 (0.40)(2.4)(1)(100)$
$= 58$

LET $Q = R_A^z$

EARTHQUAKE EFFECT:

$Q_x = Q_y = \frac{1}{2} \frac{F_W (10)}{8} = 36$

$Q_z = \frac{1}{4} (.6) F_W = \frac{1}{4}(.6) 58 = 8.6$

$Q_{E\ MAX} = \sqrt{(36)^2 + (36)^2 + (8.6)^2}$
$= 51.6$

DEAD LOAD EFFECT:

$Q_D = \frac{1}{4} W = 25$

$Q = 0.75 Q_D - Q_{E\ MAX}$ (EQ. VII-26)
$= 0.75(25) - 51.6$
$= -32.9$

ASSUME SPECIFIED WORKING STRESS VALUE OF BOLT = 25

WORKING STRESS STRENGTH OF BOLT
(1.33 TIMES BASIC WORKING STRESS, AISC SEC. 1.5.6 (540))
$= 1.33 (25)$
$= 33.3 > 32.9$

Figure VII D-2. Comparison of strength and working stress methods.

D. HORIZONTAL FORCES FOR SPECIAL SYSTEMS

This section presents techniques to calculate horizontal earthquake induced forces on resiliently mounted equipment, buried, surface mounted and elevated tanks, elements immersed in liquid, and tank and retaining walls.

RESILIENT MOUNTED EQUIPMENT

Vibration isolation systems are examples of structures, equipment and piping that are not rigidly mounted.

For resilient mounted equipment, the force F_s or F_w should by multiplied by the factor a_c, where:

F_s^R (resiliently mounted equipment) = $F_s \; a_c$

a_c = 1 for systems with a seismically activated restraining device; also for systems with an elastic restraining device, provided that

$$0.6 > \frac{T_c}{T} > 1.4$$

a_c = 2 for systems with an elastic restraining device mounted on the ground or on a slab on the ground

a_c = 2 minimum for systems with an elastic restraining device when

$$0.6 < \frac{T_c}{T} < 1.4$$

T_c = the fundamental period of the component and its attachment

$$= 0.32\sqrt{\frac{W}{K}} \qquad \text{(VII-6)}$$

where:

K = the stiffness of the equipment support attachment, determined in terms of load per unit deflection of the center of gravity (lbs./in.) as follows:

For stable resilient attachments, K = spring constant.

For other resilient attachments, K = slope of the load/deflection curve at the point of loading.

T = the fundamental period of the building in which the system is mounted

The equipment amplification factor, a_c, is taken directly from ATC 3-06. This factor takes into account the possible in-phase response of the equipment and the building in which it is mounted. Theoretically, the amplification from this phenomenon could be as high as a factor of 25. ATC 3-06 has reduced the factor to 2 because the damping and building period vary during the earthquake. In addition, the factor of 25 is based on an approximation not completely correct. However, values of $\frac{T_c}{T}$ (defined in the criteria) between 0.8 and 1.2 should be avoided if possible as a factor of safety. These criteria recommend the use of a_c for resiliently mounted equipment as it is the only type which normally has a natural frequency low enough to be in the same range as that of the building in which it is mounted.

SURFACE MOUNTED OR IN-GROUND TANKS

This subsection is concerned with calculating earthquake induced forces on surface mounted and buried tank shells, roofs and contents. The techniques are applicable to tanks in which the tank base responds with the surrounding and supporting soil.

The seismic induced forces exerted on surface mounted on in-ground tanks should be calculated in accordance with the following requirements:

Strength Design

Horizontal shear F_s^{CT} (circular tank) or F_s^{RT} (rectangular tank) at the base of the tank shell, in pounds:

$$F_s^{CT} = \frac{1.2\ C_s}{R}\left[A_a\ (W_t + W_r + W_1^{CT}) + \frac{C_1}{2}\ W_2^{CT}\right] \quad \text{(VII-7)}$$

$$F_s^{RT} = \frac{1.2\ C_s}{R}\left[A_a\ (W_t + W_r + W_1^{RT}) + \frac{C_1}{2}\ W_2^{RT}\right] \quad \text{(VII-8)}$$

with the tank wall perpendicular to the direction of the earthquake acceleration.

Bending moment M_s^{CT} (circular tank) or M_s^{RT} (rectangular tank) just above the bottom of tank shell, in foot-pounds:

$$M_s^{CT} = \frac{1.2\ C_s}{R}\left[A_a\ (W_t\ X_t + W_r\ H_r + W_1^{CT}\ X_1^{EBP}) + \frac{C_1}{2}\ W_2^{RT}\ X_2^{EBP}\right] \quad \text{(VII-9)}$$

$$M_s^{RT} = \frac{1.2\ C_s}{R}\left[A_a\ (W_t\ X_t + W_r\ H_r + W_1^{RT}\ X_1^{EBP}) + \frac{C_1}{2}\ W_2^{RT}\ X_2^{EBP}\right] \quad \text{(VII-10)}$$

with the tank wall perpendicular to the direction of the earthquake acceleration,

where:

W_t = weight of the tank shell, in pounds

W_r = weight of the tank roof plus snow load, in pounds

W_1^{CT} = effective hydrodynamic weight of circular tank contents which move in unison with the tank shell, in pounds, as shown in Figure VII-5

W_1^{RT} = effective hydrodynamic weight of rectangular tank contents which move in unison with the tank shell, in pounds, as shown in Figure VII-5

W_2^{CT} = effective hydrodynamic weight of first mode of sloshing contents of the circular tank, in pounds, as shown in Figure VII-5

W_2^{RT} = effective hydrodynamic weight of first mode of sloshing contents of the rectangular tank, in pounds, as shown in Figure VII-5.

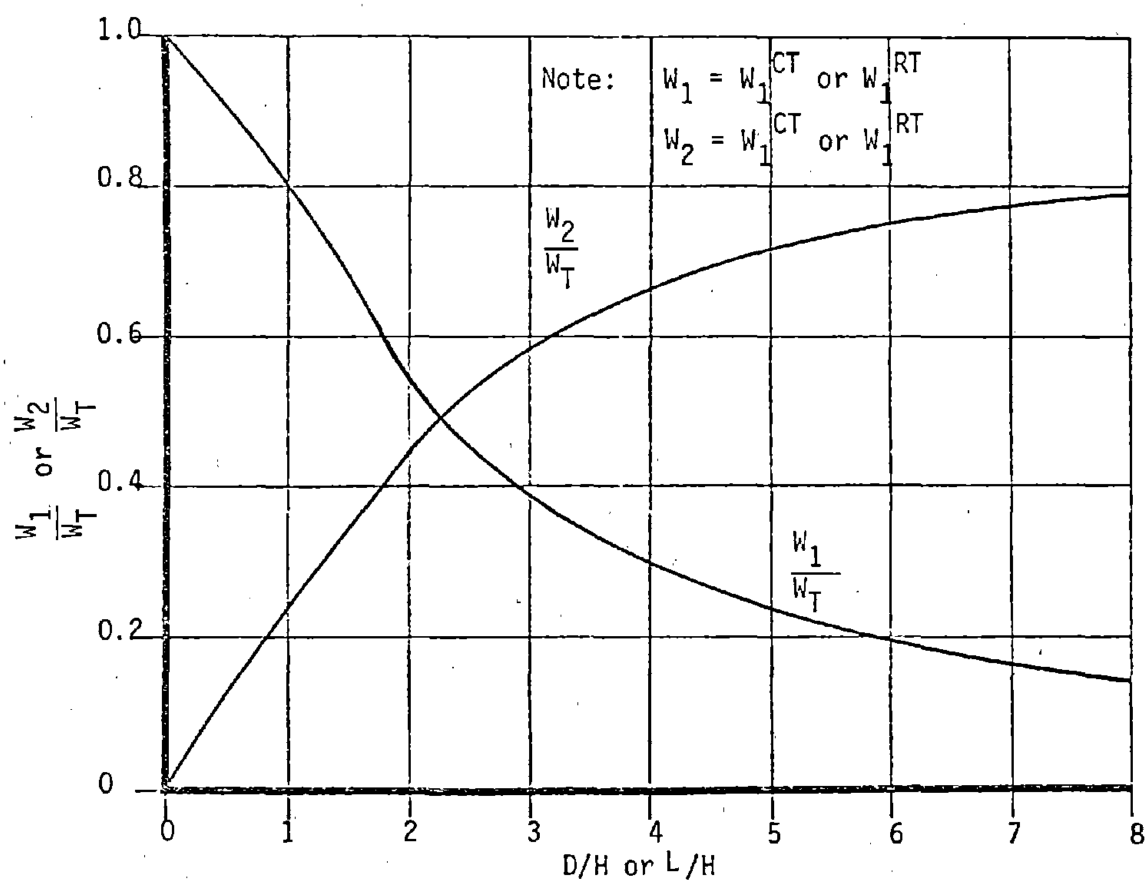


Figure VII-5. Curves for Obtaining Factors W_1/W_T and W_2/W_T for the Ratio D/H or L/H (095).

W_T = total weight of tank contents

$$= \frac{\pi D^2}{4} H\ 62.4 \text{ for a circular tank, in pounds} \qquad \text{(VII-11)}$$

$$= LBH\ 62.4 \text{ for a rectangular tank, in pounds} \qquad \text{(VII-12)}$$

where:

D = tank diameter, in feet (circular tank)

H = maximum water depth, in feet

L = tank length, in feet (rectangular tank)

B = tank width, in feet (rectangular tank)

Note: B should be interchanged with L to calculate forces on tank side walls.

X_t = height of center of gravity of W_t from bottom of tank, in feet

H_r = height of center of gravity of tank roof, in feet

X_1^{EBP} = height of center of gravity of W_1^{CT} or W_1^{RT} to bottom of tank, in feet (Excluding Bottom Pressure, EBP) as shown in Figure VII-6

X_2^{EBP} = height of center of gravity of W_2^{CT} or W_2^{RT} to bottom of tank, in feet (Excluding Bottom Pressure, EBP) as shown in Figure VII-6

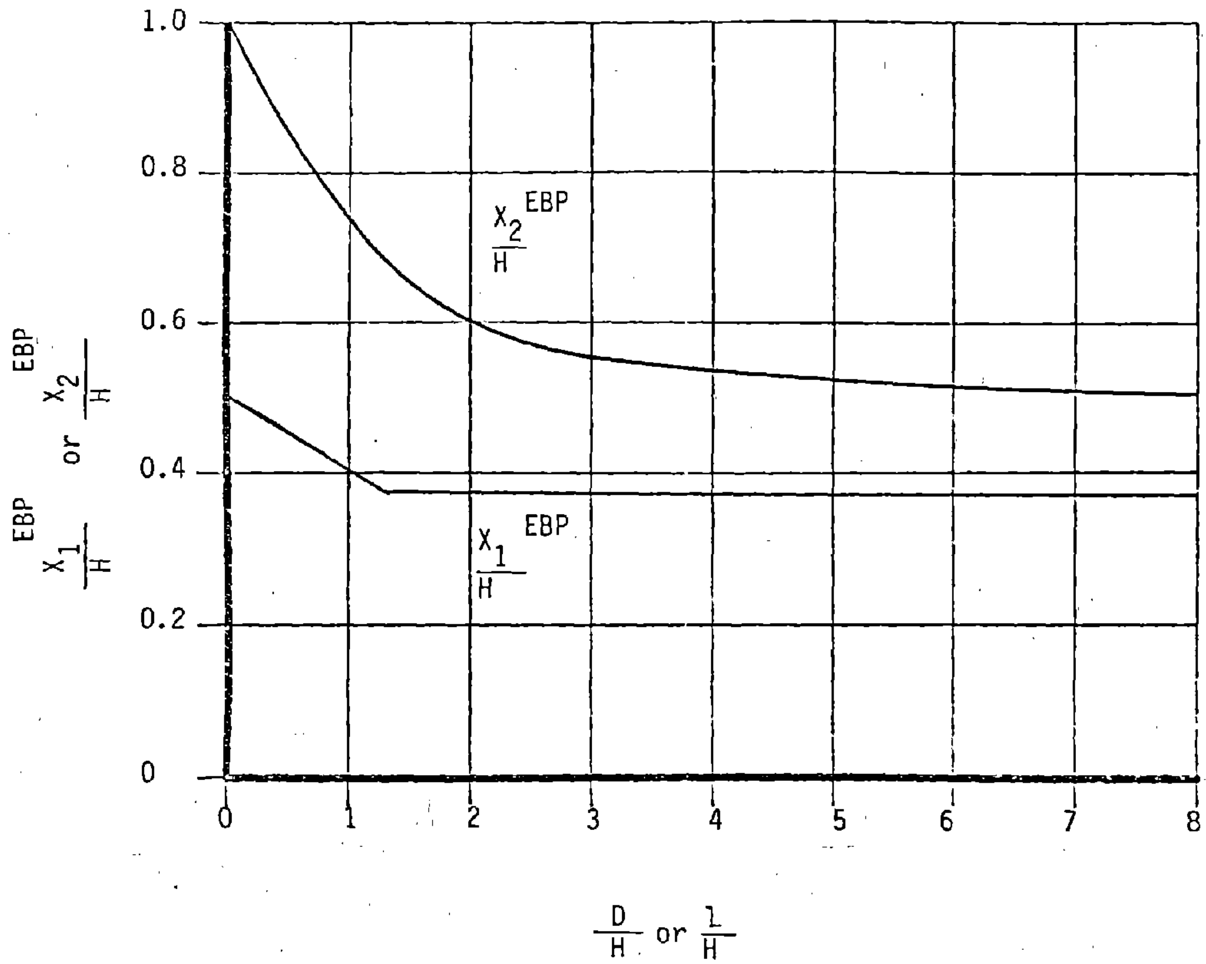


Figure VII-6. Curves for Obtaining Factors X_1/H and X_2/H for the Ratio D/H or L/H (Excluding Bottom Pressure, EBP) (095)

$C_1 = \frac{A_v S}{T^{2/3}}$ (C_1 need not be greater than 2.0 A_a and may be reduced to $\frac{2.50\ A_v S}{T^{4/3}}$; refer to page VII-35 for an explanation of the limits set.) (VII-13)

T = fundamental sloshing mode, period in seconds
$= K_p D^{1/2}$ or $= K_p L^{1/2}$ (VII-14)

where:

K_p = factor based on $\frac{D}{H}$ or $\frac{L}{H}$, as shown in Figure VII-7

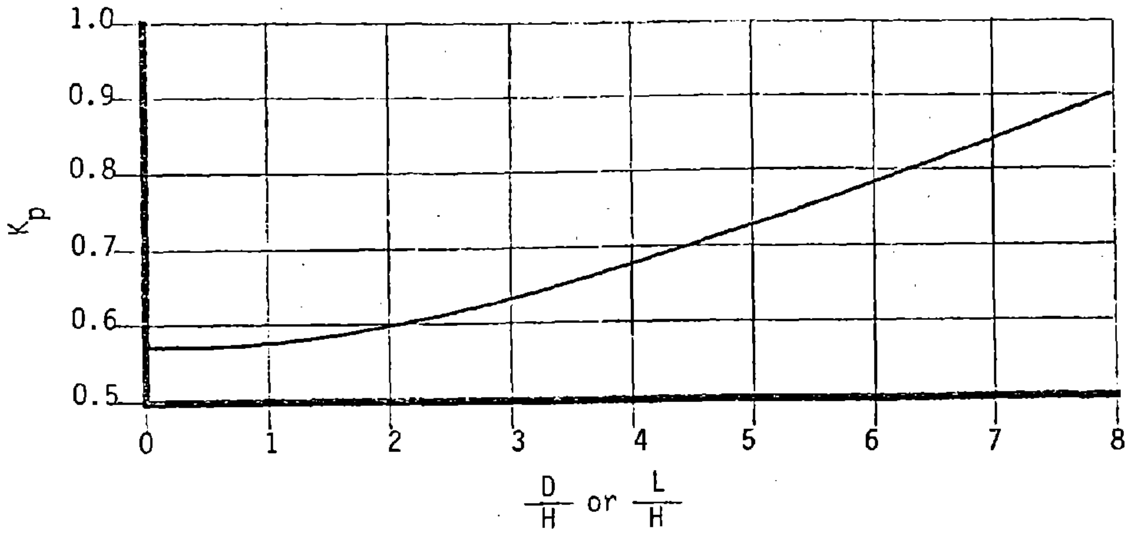


Figure VII-7. Curves for Obtaining Factor Kp for the Ratio D/H or L/H

R = response modification factor = 2.0 for anchored flat bottom tanks; 1.6 for unanchored flat bottom tanks

S = soil factor

where:

A soil profile with the following characteristics shall have an S = 1.0:

1. Rock of any characteristic, either shale-like or crystalline in nature. Such material may be characterized by a shear wave velocity greater than 2,500 feet per second, or

2. Stiff soil conditions where the soil depth is less than 200 feet and the soil types overlying rock are stable deposits of sands, gravels or stiff clays.

A soil profile with the following characteristics shall have an S = 1.2: Deep cohesionless or stiff clay conditions, including sites where the soil depth exceeds 200 feet and the soil types overlying rock are stable deposits of sands, gravels or stiff clays.

A soil profile with the following characteristics shall have an S = 1.5: Soft-to-medium-stiff clays and sands, characterized by 30 feet or more of soft-to-medium-stiff clays with or without intervening layers of sand or other cohesionless soils.

In locations where the soil properties are not known in sufficient detail to determine the soil profile type or where the profile does not fit any of the three types, S shall be equal to 1.2.

A_v = effective peak velocity-related acceleration as shown on Figures VII-3 and VII-4, as a percent of gravity

C_s = seismic coefficient as in typical design as shown on Table VII-1, except that C_s should not be less than 1.33 (C_w of 1.1)

The horizontal earthquake induced force on an exterior wall of a rectangular tank when the wall is perpendicular to the earthquake motion is equal to one-half of the horizontal shear, F_s^{RT}, at the base of the tank shell, acting at the elevations X_1 and X_2; this force is evenly distributed horizontally along the wall.

The horizontal earthquake induced force on a rectangular tank interior wall perpendicular to the earthquake motion is equal to F_s^{RT} at the elevations X_1 and X_2 and is evenly distributed along the wall.

Horizontal shear, F_s, in pounds at the base of the structure, including the tank bottom and supporting structure:

$$F_s = F_s^T + \Sigma F'_s \qquad \text{(VII-15)}$$

where:

$$F^T_s = F^{CT}_s \text{ or } F_s^{RT}$$

Overturning moment, M_s, in foot pounds at the base of the structure, including the tank bottom and supporting structure:

$$M_s = M_s^T + F_s^T h_x + \Sigma F'_s h_n \qquad \text{(VII-16)}$$

where:

F'_s = sum of force components of tank support above the base of the structure, based on F_s for each component, in pounds

h_x = height of bottom of tank above the base of the structure (point about which tank would rotate in failure), in feet

$F'_s h_n$ = summation of moments in foot pounds of each component F_s, in pounds, times the height of each (h_n) above the base, in feet.

Except that in calculating M_s^T, X_1^{EBP} and X_2^{EBP} shall be replaced by X_1^{IBP} and X_2^{IBP}, where:

X_1^{IBP} = effective height of the center of gravity of W_1^{CT} or W_1^{RT} to the bottom of the tank, in feet (Including Bottom Pressure, IBP), as shown in Figure VII-8.

X_2^{IBP} = effective height of the center of gravity of W_2^{CT} or W_2^{RT} to bottom of the tank, in feet (Including Bottom Pressure, IBP), as shown in Figure VII-8.

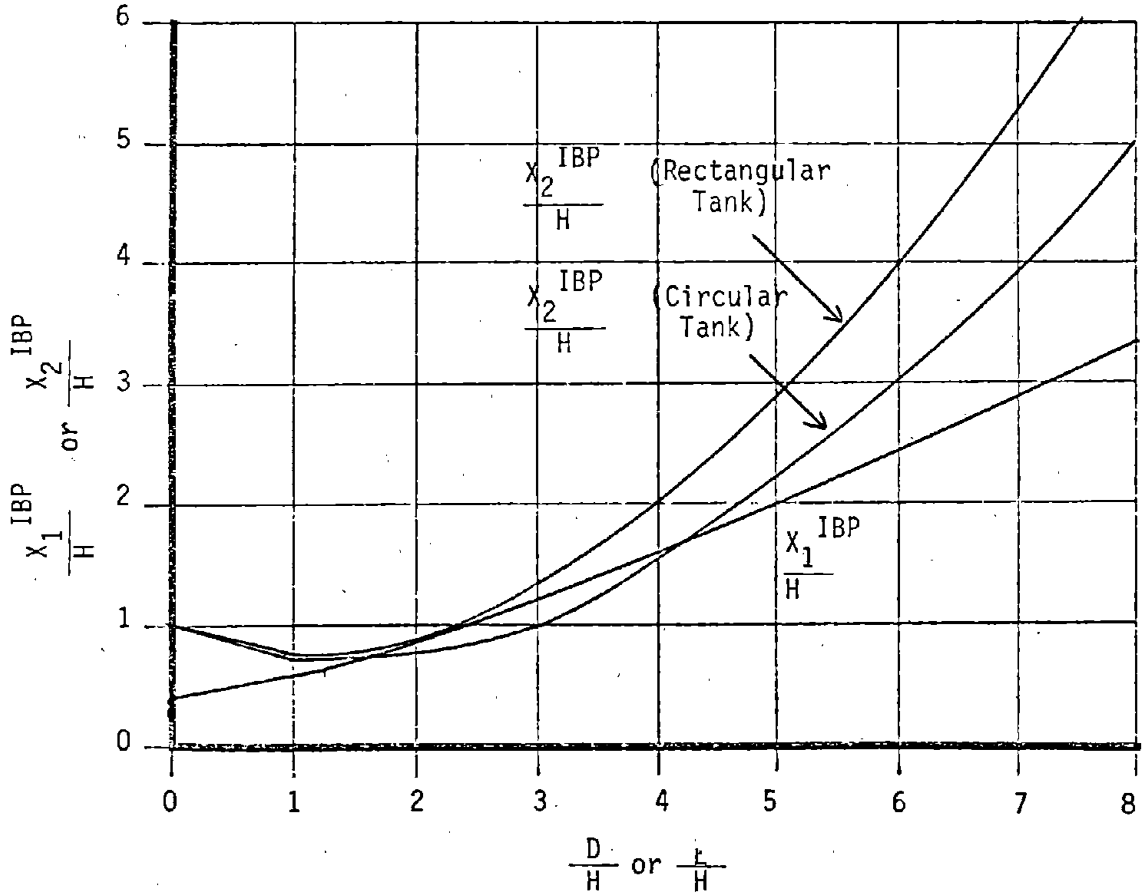


Figure VII-8. Curves for Obtaining Factors X_1/H and X_2/H for the Ratio D/H or L/H (Including Bottom Pressure, IBP) (095).

The resisting moment for unanchored steel bottom tanks should be designed in the manner described in AWWA D-100 "Standard for Welded Steel Tanks for Water Storage" (190), i.e., considering the weight of the tank shell and water acting with it.

The resisting moment for anchored steel tank/foundation structures and concrete tanks, where the tank wall/bottom responds as a single structure, will be determined using the load combinations shown in Sections C and D of this chapter.

Working Stress Design

The same formulae for working stress design shall be used, substituting the subscript w for s.

The Surface Mounted or In-Ground Tank design criteria are based primarily on work performed by Housner (095), with refinements from the AWWA "Standard for Welded Steel Tanks for Water Storage" (190) and "Basis of Seismic Design Provisions for Welded Steel Oil Storage Tanks" by Wosniak et al. (194), and developed to be in accord with ATC 3-06 (141).

Housner modelled a tank and liquid contents responding to earthquake motions as a rigid tank with two weights suspended inside it, W_1^T and W_2^T. W_1^T is connected to the tank walls by two rigid members at height X_1 from the bottom, representing the water moving in unison with the tank wall. W_2^T is connected to the tank wall by two springs at height X_2 from the bottom and represents the weight of water moving back and forth (sloshing) across the tank in its natural frequency. The model is illustrated in Figure VII D-3.

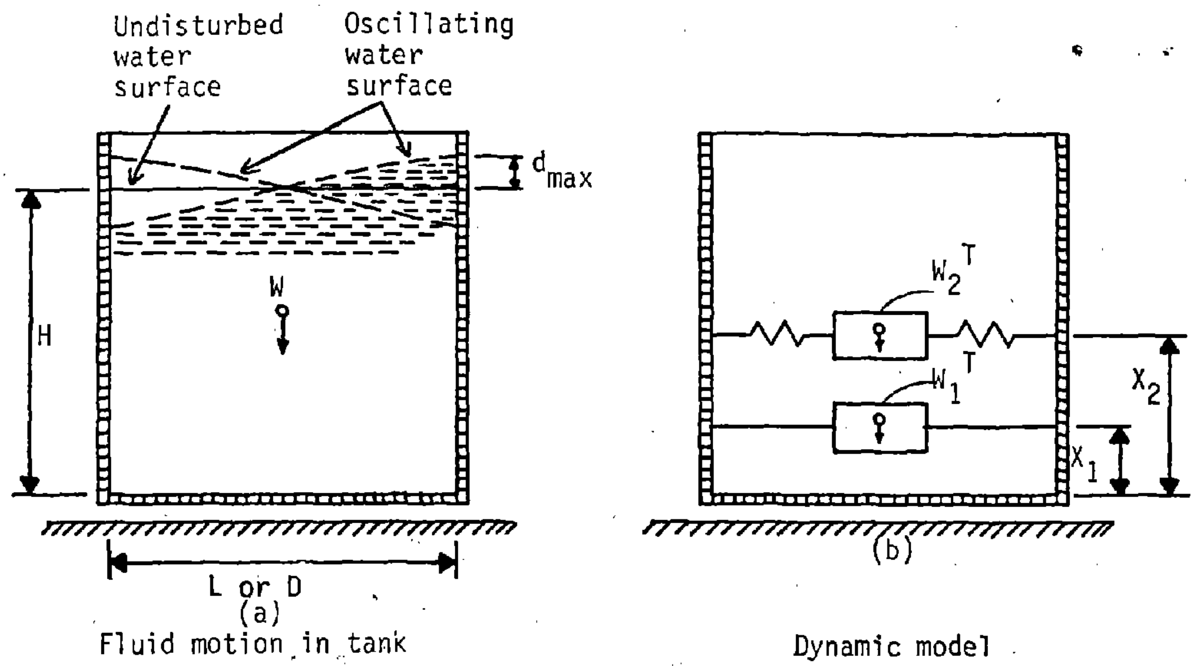


Figure VII D-3. Housner tank model (095).

A_a is used to represent the high frequency range of earthquake motions within which the tank and W_1^T respond. A_v is used to represent the lower frequency earthquake motions approximating those of the sloshing water. It can be seen that the driving force, C_1, is a function of both A_v and T; it is thus period dependent. A plot of the term approximating the acceleration of the rigid response portion of the tank and contents is shown as the horizontal portion of the ATC 3-06 plot in Figure VII D-4. This is the same level of acceleration to which rigid equipment is subjected as discussed in Section C. A curve approximating the acceleration of the sloshing component of the equation, a function of C_1, is shown in Figure VII D-4 to decrease as the period increases. This same curve is used to approximate the acceleration to which an elevated tank is subjected, the details of which are included further on in the text. As the structure's period decreases, it approaches the design acceleration of rigid equipment.

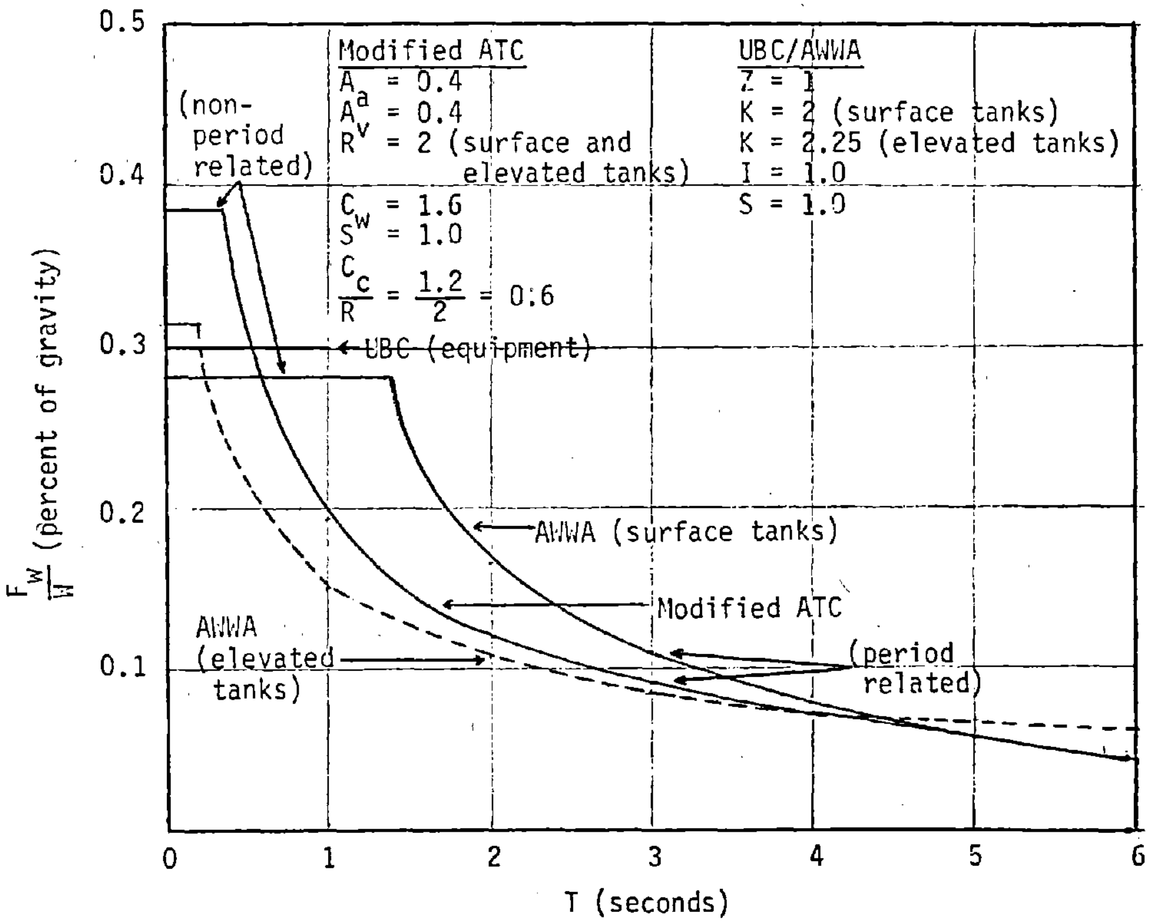


Figure VII D-4. Levels of design accelerations.

Equations VII-7,8,9 and 10 are in the same form as that found in AWWA D-100, with several modifications. The AWWA D-100 equation with nomenclature changed for consistency representing the same load effect as for the bending moment, M_s^{CT}, equation VII-9, is as follows:

$$M_s^{CT} = ZK\, [\, 0.14\, (W_t\, X_t + W_r\, H_r + W_1^{CT}\, X_1^{EBP}) + C_1\, SW_2\, X_2] \qquad \text{(VII D-11)}$$

In the first term, ZK (0.14) has been modified to 1.2 C_s A_a/R, with $1/R$ replacing K and the pseudo-acceleration A_a replacing Z. In the second term, ZK C_1S has been modified to 1.2 C_s $C_1/2R$, with S being included in the variable C_1 in the criteria presented herein. C_1 in the AWWA equation is calculated somewhat differently from that included in this report. The C_1 presented here is based on the ATC 3-06 representation of the design spectra. It was used because it was believed to be the most recent information available.

Plots of the AWWA values for the first and second terms are shown in Figure VII D-4 for comparison purposes. The first term of the criteria presented here is more conservative than AWWA's value, assuming comparable zone coefficients A_a, A_v and Z, and designating a C_w of 1.6 (intermediate shutdown is acceptable). (Note: C_w is used rather than C_s because AWWA uses working stress design.) AWWA's values are more conservative in the sloshing mode, again designating a C_w of 1.6. When combined, the approach presented here is approximately the same as that taken by AWWA. Note: The curves shown on Figure VII D-4 are not related to the concept of reducing the acceleration to which a structure is subjected by increasing its rigidity, as previously discussed. The acceleration is maximized at a structural response between frequencies of approximately 3 and 9 cycles per second (.35 to .11 seconds period) and decreases as the frequency increases. That acceleration reduction occurs within the period bounds of the horizontal line in Figure VII D-3 and is not taken into account in the criteria.

A superscript, EBP (excluding bottom pressure), has been added to the reaction height (X_1 and X_2) of W_1^T and W_2^T, denoting exclusion of bottom pressure considerations which do not affect the tank shell. X_1^{EBP} and X_2^{EBP} are used in AWWA's equation but are not noted as such. A different superscript will be discussed later.

In most instances, C_s would be assigned a value of 2. A minimum C_s of 1.33 (C_w of 1.1) (comparable to AWWA's 50% increase in K for more essential facilities) may be used for surface mounted steel tanks if they are non-essential. A C_s of 3 increases the design forces by 50% above the AWWA design standard and would be required only in critical situations.

K_p is a coefficient included for simplification as follows:

$$K_p^2 = \frac{1}{3 \tanh 3.68 H/D} \qquad \text{(VII D-12)}$$

Equations for determination of forces in rectangular tanks are based on Housner's work (095). The only difference in calculations of forces on the two types of tanks as presented is found in the calculation of the weight of the tank contents. The authors of this report have simplified the procedure by assuming that the plot shown for the determination of W_1, W_2, X_1 and X_2 is the same for rectangular and circular tanks, except for X_2^{IBP}, as shown in Figure VII-8. The equations for W_1 and X_1 are identical for circular and rectangular tanks. The plot for W_2/W_T would be slightly higher for rectangular tanks if it were plotted separately. X_2^{EBP}/H would be slightly less for rectangular tanks than for circular tanks. Separate plots are not included for rectangular tanks because the difference from the corresponding values for circular tanks is considered minimal. Also, the loading from the sloshing factor is small compared to the impulsive force. The K_p for rectangular tanks is slightly larger than for circular tanks, giving them longer periods for comparable D/H and L/H values. This would reduce C_1 for rectangular tanks, making the recommended criteria's approach conservative for these tanks. For a discussion of the equations used for the development of Figures VII-5,6 and 8, refer to Housner's work (095).

The soil factor, S, is taken directly from ATC 3-06. The purpose of this factor is to represent the amplification of long period earthquake motions on soft soils driving the long period sloshing of the tank contents. Soft soils also may filter out short period motions which in effect may reduce the acceleration on short period structures. This short period phenomenon was not taken into account in the criteria as its effect was judged not to be of adequate significance to justify the added complexity introduced by its consideration.

Determination of the force on a single wall of a rectangular tank is determined simply by dividing the total lateral shear force, F_s^{RT}, by two. This determination is apparent in Housner's development of the tank response model where W_1^T and W_2^T are attached to both sides of the rectangular tank walls which are perpendicular to the earthquake motion. The water contained in the tank cannot produce a tensile force on the tank wall which is accelerated away from the water mass. The induced force is, however, transferred to the tank by a reduction in the static water pressure exerted on the tank wall.

Equations VII-7,8,9 and 10 define loadings exerted on the tanks' walls and do not take into account earthquake induced loading on the tank bottom. This is an important consideration in the design of the tank foundation and support structure. Equations VII-15 and 16 take these forces into account by increasing the reaction heights X_1 and X_2 of the weights W_1^T and W_2^T, respectively. Housner (095) developed the equations for X_1^{IBP} and X_2^{IBP}, including bottom pressure, that are used to plot Figure VII-8. X_2^{IBP}/H for rectangular tanks is plotted separately in Figure VII-8 because of its significant increase over that for circular tanks.

The response of the liquid in a tank to earthquake motions is that of a sloshing effect, waves of water moving back and forth across the tank, the forces of which are taken into account by the response of W_2^T. This sloshing phenomenon increases the water level at the side of the tank by a height d_{max}, in feet, above the static water level shown in Figure VII D-3. In an open tank, this may cause overtopping of the liquid or, in a covered tank, an uplift force on the tank cover if there is not adequate freeboard. The sloshing height can be calculated using an equation developed by Housner (095). and modified to be consistent with these criteria as follows:

$$d_{max} = \frac{0.61\ K_p^{\ 2}\ D}{\left(\frac{2.72\ D^{1/3}\ K_p^{\ 8/3}}{C_s\ A_v\ S} - 1\right)} \qquad \text{VII D-13}$$

where all terms are in English units and previously defined.

Shaking table testing of tanks (Clough 195) has shown the Housner approach to be reasonably reliable for determination of the lateral forces and sloshing period. Representatives of the AWWA D-100 Standard Committee have indicated that the Housner approach as included in AWWA D-100, although slightly conservative, is the most practical current approach. Clough (195) showed, however, that in determination of the maximum water surface displacement d_{max}, when values of H/D dropped below 0.4, the calculated water displacement was considerably less than that observed. With an H/D of 0.25, the calculated value was 56 percent of the observed value. Consideration should be given to providing freeboard in addition to the calculated water surface displacement in these situations if it is an important design parameter. In open tanks, such as clarifiers and aeration basins, lack of adequate freeboard may result in over-topping.

An alternative approach to the procedure for analyzing liquid storage tanks developed by Housner (095), adopted by AWWA D-100 (190) and presented in this report, has been reported by Veletsos and others (196,197). Housner's model assumes the tank to be rigid. A cylindrical tank shell, however, has been shown to respond as a flexible structure (198, 195). This may increase the seismic effect on the structure, the effective earthquake imput acceleration, which is frequency dependent. One of the effects of flexibility, as previously discussed, is that as the structure's natural frequency decreases below 33 cps, to about 9 cps, the effective acceleration to which the structure is subjected increases. Such is the case when considering flexible tanks rather than rigid ones. Veletsos' work considers the effect of this flexibility on tank design. This, however, would only effect the impulsive force consideration, as the frequencies involved with convective forces are much lower. This increased acceleration has been included in AWWA D-100 and is accounted for in this report by including the increased accelerations between 33 cps and 2.5 cps in the force equations.

ELEVATED TANKS AND STRUCTURES

Techniques are presented in this sub-section for elevated tanks and structures for which the structure responds at its own natural frequency.

Elevated tanks should be designed by the following requirements:

Strength Design

$$F_s^E = \frac{0.6\ C_1\ C_s\ W}{R} \qquad \text{(VII-17)}$$

where:

F_s^E = force applied at the center of gravity of the tank and contents

C_1 as defined previously in Section B

C_s = seismic coefficients, as shown in Table VII-1, with the exception that C_s should not be less than 2.0 (C_w of 1.6)

R = response modification factor equalling 2.0 for braced tanks, or 1.50 for pedestal tanks

T = calculated fundamental period of vibration of tank system, in seconds

W = lumped weight of tank or structure, tank contents, and effective weight of tower, in pounds. Assume 5% eccentricity of center of mass for calculation of horizontal forces

Working Stress Design

The same equation applies for working stress, except that the subscript changes from s to w.

The horizontal induced forces on elevated tanks are computed in exactly the same manner as those induced on the long period sloshing liquid contents, as modeled by the weight W_2^T for surface mounted tanks. AWWA D-100 does not allow a reduction in the structure coefficient K for elevated non-essential use elevated tanks as it does with surface mounted tanks. If a C_w of 1.6 were used (working stress design), these criteria are slightly more conservative than AWWA's as shown in Figure VII D-4. This allows more flexibility in the design, allowing the designer to choose a design level C_w of 1.6 or 2.4 (C_s of 2.0 or 3.0), not possible with AWWA's criteria. A C_w value less than 1.6 (C_s of 2.0) is not recommended for use in determining lateral forces on elevated tanks, unless completely isolated from other systems and non-essential in usage. Its use may result in collapse of the tank support structure, causing damage below or excessive hazard to human safety.

The sloshing mode has not been taken into account in elevated tank design, as its effect is small as compared to the portion of the liquid responding with the tank structure. The designer may choose to account for the sloshing mode by using the criteria presented for surface mounted tanks. Resonance of the sloshing liquid and the tank supporting structure response should be avoided. This may be a problem when designing a very high small diameter tank. A factor for torsion has been included (140) by requiring a 5 percent eccentricity placement of the lumped weight of the tank. This could be important if the support system has a primary resisting system weak in torsion.

ELEMENTS IMMERSED IN WATER OR SEWAGE

Increased Effective Weight

This subsection presents a technique for calculating the effective weight, W, of structures immersed in water which, in turn, is to be used in conjunction with the criteria presented previously in this chapter. Examples of elements immersed in water include clarifier center wells, baffles, aerators and piping.

The effective weight, W, of a structure immersed in water or sewage when responding to earthquake motions is as follows:

$$W = W_S + W_T + w_a H \qquad \text{(VII-18)}$$

where:

W_S = weight of the structure, in pounds

W_T = weight of water contained in the structure, in pounds

If the height of the structure containing water is less than 0.75 times its diameter, consideration may be given to the effect of sloshing within the structure, as presented in the tank design section.

w_a = weight of the liquid immediately outside and responding with the structure per lineal foot of height of the structure, in pounds per lineal foot

$$w_a = \alpha_{wr} \lambda_W \pi r_o^2 \qquad \text{(VII-19)}$$

$\lambda_W \pi r_o^2$ = weight per lineal foot of height of a cylinder of liquid with the radius r_o, in pounds per lineal foot

α_{wr} = added weight ratio being 1.25 for flat two-dimensional structures such as a baffle plate vibrating normal to its axis (or a prismatic structure acting similarly)

α_{wr} = added weight ratio for cylindrical or cylindrical-like structures as shown in Figure VII-9

λ_W = unit weight of liquid, in pounds per cubic foot

r_o = radius of the structure at height where α_{wr} is determined, in feet (or ½ of the projected width)

H = height of structure that is submerged, in feet

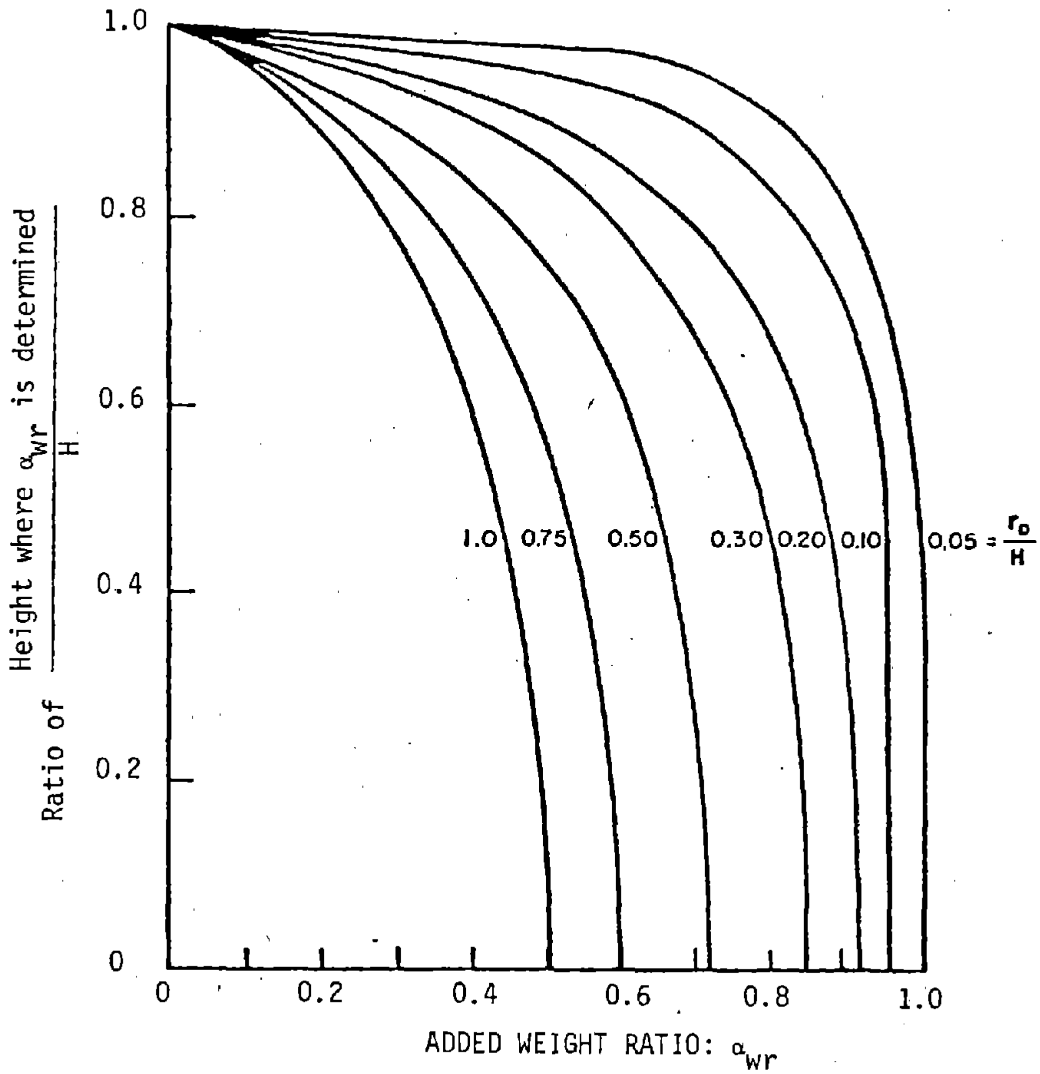


Figure VII-9. Added Weight Representing Hydrodynamic Effects (150)

In part, the lateral forces induced on immersed structures are accounted for by assuming that the effective weight of the structure includes the weight of the water surrounding and responding with it. The factor presented for two dimensional structures is based on the assumption that the structure is flexible or mounted flexibly. This information is based on lab test results with a conservative analysis. The damping of a structure immersed in liquid is assumed to be 2 percent (001). The criteria presented for cylindrical 3 dimensional structures are developed with the aid of results of analyses of two recently constructed water intake towers (150).

Convective Force From Sloshing

This subsection presents a method for calculating the lateral force on an immersed structure from the drag force of water sloshing horizontally across the tank.

Strength design -

The drag force F_s^D can be calculated as follows:

$$F_s^D = C_p\, A\, u_s^2 \qquad \text{VII-20}$$

where:

F_s^D = force applied to the centroid of the projected area of the structure, in pounds

C_p = drag coefficient; 2 for flat plates, 1 for cylindrical shapes

A = projected area perpendicular to the direction of sloshing, in square feet

u_s = horizontal velocity of sloshing, in feet per second

$$u_s = \frac{10\, K_p\, D^{\frac{1}{2}}\, C_s\, C_1}{R} \qquad \text{VII-21}$$

where all the above terms have been previously defined.

Working stress design -

The same equations apply for working stress, except the subscripts change from s to w.

Combination of Forces From Sloshing

F_s^D is to be directly additive to the direct forces from the effective weight determined from equation VII-18.

The drag force equation VII-20 is a simplification of the following equation found in most fluid mechanics texts:

$$F_D = C_D \tfrac{1}{2} \rho \; u^2_\infty A \qquad \text{VII D-14}$$

The values designated for C_D are based on the assumption that the Reynolds numbers are in the turbulent flow range. ρ is the fluid density. Using English units, $\frac{1}{2} \rho$ is rounded off to 1. u_∞ is the velocity of water far enough away from the immersed element so that it has no effect on the velocity. This assumption seems valid for small immersed elements. When larger elements are encountered, e.g., clarifier center wells, they may effect the sloshing flow pattern throughout the tank. However, higher modes of sloshing would probably become more predominant, decreasing the net horizontal velocity. There has been little empirical work in this area.

The equation for horizontal velocity, u, was derived from work done by Housner (095), with additions made from the approach adopted from ATC 3-06 and used in this report.

Several assumptions were made in order to simplify the equation for u. The equation was developed using the fundamental sloshing period of circular tanks. The equation approximates the horizontal velocity of sloshing water in rectangular tanks by substituting the rectangular tank length L for the circular tank diameter D. Higher modes of sloshing response would have reduced velocities. The equation is simplified to calculate the maximum water velocity. This occurs, according to Housner's work, on the water surface at the center of a circular tank. The velocity will decrease as the depth increases. Jacobsen [199; from Housner (095)] has shown that there is no water movement perpendicular to the direction of the earthquake motions. Essentially, circular tanks respond as a series of narrow rectangular tanks of varying lengths placed side by side. As the velocity is measured further away from the center of a circular tank (perpendicular to the water motion), the effective length of the narrow rectangular tank decreases, as does the velocity.

Sloshing is basically a mass of water moving back and forth across a tank responding to an earthquake in a manner similar to that of a swinging pendulum. As the mass of water approaches the tank wall, its static head increases (d_{max}, previously discussed) increasing its potential energy. As the mass moves towards the tank center, its velocity increases (increased kinetic energy). The equation calculates this maximum velocity.

To the authors' knowledge, there have been no shaking table tests to measure the sloshing water velocity. However, as previously discussed, empirical results have been compared with calculated maximum wave heights, d_{max}, with significant inconsistencies observed at low H/D values. The force, F_s^D, calculated using this procedure should be directly added to other earthquake induced forces acting in the same direction and, conceivably, at the same time.

RETAINING WALLS

Retaining walls are important in water and sewage treatment plant design. Examples include clarifiers, pump pits, clear wellsand reactor tank walls. This subsection presents a technique to calculate the increased lateral earth pressure induced by earthquakes in conjunction with a static lateral earth pressure determination chosen by the design engineer for tank, vault and retaining walls.

Working Stress Design

If A_a C_w is greater than 0.17, the increase in active earth pressure from seismic accelerations should be taken into account as follows:

The increase in the lateral active earth pressure force, F_w^W, due to seismic accelerations should be calculated as follows:

$$F_w^W = \frac{\lambda H^2 A_a S_c C_w}{6 \tan \phi} \left(1 + \frac{6i}{100}\right) \qquad \text{(VII-22)}$$

where:

F_w^W shall act at a point 0.6H above the base of the wall

λ = unit weight of backfill

H = height of fill retained by the wall

A_a = seismic coefficient representing the Effective Peak Acceleration as previously defined in Section A

S_c = site coefficient where:

S_c = 1 for bedrock, well consolidated rock-like soils; may include ridges, hills and mountains

S_c = 1.25 for dense soils with low moisture content (alluvium); may include valleys and areas bordering foothills, etc.

S_c = 1.5 for water saturated gravels, sands and clays, including man-made saturated hydraulic fills and bay mud

C_w = seismic coefficient for working stress design as shown in Table VII-2

i = angle of slope behind the retaining wall to horizontal, in degrees

ϕ = angle of internal friction of the soil

The maximum elevation of the water table shall be below the bottom of the wall (assumption made for simplification, see discussion for explanation).

An allowance for excluding seismic design below a stated modified acceleration, A_a C_w, is included as the factor of safety against sliding which may be reduced from 1.5 for static design to 1.15 for earthquake design because of the temporary nature of earthquake forces. This cut off is based on a static earth-pressure coefficient, K_a, of 0.25 (200). This is consistent both with allowing a 33 percent increase in allowable stress when using working stress design, and with reducing the dead load factor when combining loadings when using strength design for seismic loadings.

The design engineer should refer to Section G for a discussion of combination of earthquake and other loadings. A reduction of moment resisting dead loads is required, a crucial consideration for independent retaining walls.

The equation is developed below for working stress design because this is typically used when working with soil loadings. A discussion of convert-ing the procedure to strength design will follow.

Equation VII-22 is a simplification of an equation developed by Okabe (1926) and Mononobe (1929), which was developed from the Coulomb static lateral soil pressure theory for dry cohesionless soils, a standard approach.

The equations included in this development are based on several assumptions (200):

a. Minimum horizontal yielding of the wall to develop active earth pressures (.001 H).

b. When the minimum active earth pressure is developed, a soil wedge behind the walls is at its threshold of failure with the maximum shear strength developed along the wedge's lower edge.

c. The soil wedge acts as a rigid body. If the retaining wall does not yield, not allowing the maximum shear strength of the soil to develop, the lateral earth pressure may be substantially greater.

The basic Coulomb equation for calculating the force resulting from the static lateral active soil pressure is as follows (201):

$$F_W^{W\ (static)} = 1/2\ \lambda\ H^2\ K_a \qquad \text{VII D-15}$$

where:

$F_W^{W\ (static)}$ = force resulting from the active static soil pressure per unit width; assume a triangular distribution through depth with apex at the top of the wall

λ = unit weight of soil

H = height of fill retained by wall

K_a = active earth pressure coefficient where:

$$K_a = \frac{\cos^2(\phi-\beta)}{\cos^2\beta\ \cos(\beta+\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\ \sin(\phi-i)}{\cos(\beta+\delta)\ \cos(\beta-i)}}\right]^2} \qquad \text{VII-D-16}$$

where: ϕ = angle of internal friction of soil

δ = angle of friction between structure and soil (often assumed as zero)

β = angle of batter of the wall from vertical

i = angle of the sloping backfill from horizontal

The complete Okabe/Mononobe equation for computation of the dynamic (includes static and dynamic forces) active earth pressure coefficient, $K_{ae}{}^{m}$, for moist soil (revision of Coulomb equation) is as follows (201);

$$K_{ae}{}^{m} = \frac{(1 \pm k_v)\cos^2(\phi-\theta-\beta)}{\cos\theta\ \cos^2\beta\ \cos(\beta+\delta+\theta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\ \sin(\phi-i-\theta)}{\cos(\beta+\delta+\theta)\ \cos(\beta-i)}}\right]^2} \qquad \text{VII D-17}$$

where:

$$\theta = \tan^{-1}\left(\frac{k_h}{1 \pm k_v}\right) \qquad \text{VII D-18}$$

$$\theta' = \tan^{-1}\left(\frac{k_h}{1 \pm k_v}\cdot\frac{\lambda}{\lambda-\lambda_w}\right) \qquad \text{VII D-19}$$

λ = total unit weight of soil

λ_w = unit weight of water

k_h = horizontal seismic coefficient

k_v = vertical seismic coefficient

The force resulting from the dynamic (including both static and dynamic forces) lateral soil pressure, $F_w^{w\,(total)}$, is computed the same as the force resulting from the static lateral soil pressure $F_w^{w\,(static)}$, using equation VII D-15, except that K_{ae} replaces K_a (200).

$$K_{ae} = K_a + \Delta K_{ae} \qquad \text{VII D-20}$$

where: ΔK_{ae} = the increase in the lateral earth pressure coefficient from seismic forces

Therefore (201):

$$F_w^{W(total)} = F_w^{W(static)} + F_w^{W} + 1/2\ K_a\ \lambda\ H^2 + 1/2\ (K_{ae}-K_a)\lambda\ H^2 \qquad \text{VII-D-21}$$

and:

$$F_w^{W} = 1/2\ \lambda H^2\ \Delta K_{ae} \qquad \text{VII D-22}$$

where: F_w^{W} = the force (per unit width) resulting from the increment due to seismic loading; assume an inverted triangular distribution through depth with the apex at the base of the wall

When the soil is partially submerged, separate values of K_{ae} must be calculated. The force from static water pressure must be added to the above calculated forces.

Several assumptions to simplify the equations can be made as follows:

δ - little effect (201)

β - assume 0 for most tank walls (200)

k_v - usually increases proportionally with K_h, therefore neglect (200)

ϕ - is assumed to be 35^o (200) (a minimum value for well consolidated cohesionless soil)

i - assumed to be 0; level ground

With these assumptions, for practical purposes (200):

$$\Delta K_{ae} = 3/4\ k_h \qquad \text{VII D-23}$$

In a manner similar to the development of equation VII-10, in accordance with this report, assume that the wall responds as a rigid structure, therefore letting R=2, and, substituting C_w for C_s (equation VII-10):

$$k_h = \frac{1.2\ C_w A_a}{R} = 0.6\ C_w\ A_a \qquad \text{VII D-24}$$

Combining equation VII D-22 with equation VII D-23 and equation VII D-24:

$$F_w^{W} = 1/2\ \lambda\ H^2 \left[3/4\ (0.6\ C_w\ A_a) \right] \qquad \text{VII D-25}$$

If the following refinements are made:

A site coefficient, S_c, taken from design criteria developed by the East Bay Municipal Utility District (EBMUD) (162), accounts for the possibility of local soil instability.

The effect of increasing ϕ above 35^o can be approximated by multiplying F_w^W by $\frac{0.7}{\tan\phi}$ (162).

The effect of increasing i above 0, i.e., sloping backfill, can be approximated by adding .06 (i) (F_w^W) to the F_w^W calculated with i equal to 0.

Equation VII D-25 then becomes:

$$F_w^{W} = \frac{1/2\ \lambda\ H^2\ \left[3/4\ (0.6\ C_w\ A_a)\right]\ S_c\ (0.7)}{\tan\phi} \left(1 + \frac{6i}{100}\right) \qquad \text{VII D-26}$$

Rounding off produces equation VII-22.

If a C_w of 1.6 is used in the equation with values of A_a and S_c corresponding to comparable coefficients in EBMUD's Seismic Design Standard (162), F_w^W will be approximately 1.7 times the value calculated using EBMUD's equation. This is directly proportional to the accelerations used in each equation.

Strength Design

The same equation is applicable to strength design, except that subscripts change from w to s, and if $A_a\ C_s$ is greater than 0.21, the increase in active earth pressure from seismic accelerations should be taken into account.

Working stress design is currently used with corresponding service loading values for lateral soil pressures in retaining wall design. The increased value in the seismic portion of the force, when changing from working stress to strength design, F_s^W rather than F_w^W, is calculated directly by an increased C factor; i.e., C_s rather than C_w. The static lateral earth pressure force, $F_w^{W(static)}$, must also be increased when used in strength design as follows:

$$F_s^{W\ (static)} = F_w^{W\ (static)}\left(\frac{C_s}{C_w}\right) \qquad \text{VII D-27}$$

Two sources (113, 201) recommending criteria for seismic resistant design of quay walls and cellular cofferdams disregard any forces induced on the retaining wall by dynamic water pressure other than those considered for submerged soil. In both cases, it was believed that the water moves with the soil as a single mass. Another study (200), however cites the reliability of an analysis by Matsuo and O'Hara (1960), where the dynamic lateral soil pressures are calculated independently. The dynamic water pressure was calculated to be 70 percent of the dynamic water pressure as determined by the Westergaard theory (200). The Westergaard theory (for calculation of dynamic water pressures on the face of a concrete dam) for the total dynamic water force, P, on a vertical wall in water of depth H, is as follows:

$$P = 7/12\ H^2 \gamma_w \cdot k_h \qquad \text{VII D-28}$$

where all other units have been previously defined.

Prakash (202) has recommended that the dynamic increment of lateral earth pressure be applied at 0.45 H above the base of a rigid wall (concrete) and 0.55 H above the base of a flexible wall (steel plate). This would be indicative of the dependence of deformation of the retained earth on developing shear stress along the failure plane. He has also suggested that the height of application be dependent on the horizontal acceleration, k_h. A conservative, more widely adopted application height is 0.6 H above the base of the wall, as recommended by Seed (200).

The calculation of the increase of active lateral earth pressure due to earthquake accelerations does not take into account static lateral earth pressures as it is presented. The design engineer should use a standard method of static lateral earth pressure determination available in most soil mechanics and foundation engineering texts and combine these load effects in the usual manner given in Sections F and G.

If the backfill material is subject to liquefaction, the retaining wall should be designed to resist the static force exerted by a liquid with a density equal to the submerged backfill. Dynamic forces are not taken into account where liquefaction is the determining design factor, as liquefaction does not occur until some time after the earthquake motions have ceased (162).

E. VERTICAL FORCES

Vertical forces can be an important consideration in seismic design, such as in the calculation of compression forces on equipment legs, particularly in combination with overturning moments. A technique is presented in this section for calculating the vertical earthquake induced forces on structures, piping and equipment.

Vertical earthquake induced forces induced on elements, V_s and V_w, should be 0.6 times the horizontal earthquake induced forces, F_s and F_w, respectively, calculated using the vertical period of the system, with the following exception:

> Vertical earthquake forces need not be considered in calculation of the increased lateral earth pressure nor in combination with load effects of horizontal earthquake forces.

Static water pressure should be increased in proportion to the increase in net vertical forces.

The earthquake induced portion of "static" water pressure loading should be combined the same as other earthquake induced forces (190, 203).

The fundamental period of radial vibration of surface mounted steel shell liquid storage tanks responding to vertical earthquake motions, T_v, may be taken as (203):

$$T_v = 0.00036 \sqrt{D\ \sigma_{all}} \qquad \text{VII-D-29}$$

where:

T_v = fundamental period of radial vibration, in seconds

D = tank diameter, in feet

σ_{all} = tank wall allowable hoop stress (working stress design), in psi

Vertical forces should be considered in both an upward and downward direction with the combination of the load effects calculated in both cases.

Vertical accelerations could conceivably instantaneously reduce the effect of gravity to 0 (weightlessness). Therefore, friction between the base of the structure and the supporting surface should not be taken into account in design to resist horizontal earthquake induced loadings.

The P-delta effect (the effect of vertical forces on tall, long period structures that have deflected to an eccentric position above their supporting structure, as illustrated on Figure VII D-5) although seldom a concern in structures covered by this report, should be checked where appropriate. A detailed analysis of this phenomenon is available in ATC 3-06.

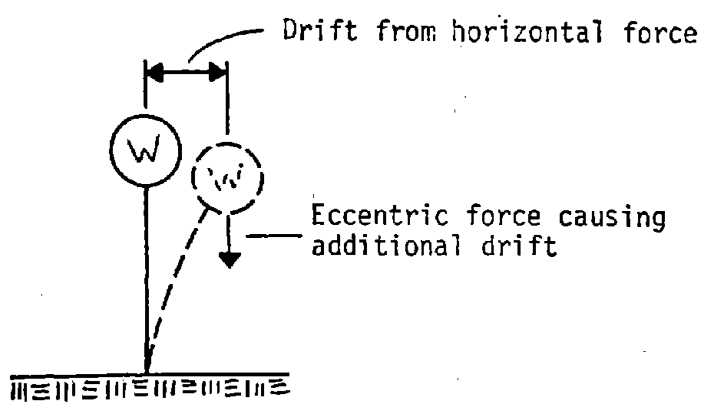


Figure VII D-5. P-delta effect.

F. COMBINATION OF EARTHQUAKE LOAD EFFECTS

This section should be used to calculate the combination of earthquake load effects including forces, moments and deflections.

The resultant maximum earthquake load effect, Q_E, (stress) should be taken as

$$Q_E \max = \sqrt{Q_x^2 + Q_y^2 + Q_z^2} \qquad \text{(VII-23)}$$

where Q_x, Q_y and Q_z are the seismic load effects with loads applied in 3 orthogonal directions of load application. The resultant earthquake load effect should then be combined with other load effects in accordance with Section G.

This method of considering loading combinations is the most conservative when the loads from different directions are approximately equal. If three equal loadings producing the same load effects on a structure are applied from the x, y and z directions, the resultant is equal to 100 percent of the first loading and 36 percent each of the second and third loadings. As the second and third loadings decrease in relation to the first, their calculated effect decreases. ATC 3-06 criteria, on the other hand, require 30 percent of the orthogonal horizontal loading and 33 percent of the vertical loading based on the response in the maximum earthquake zone to be included. While the ATC 3-06 approach may be simpler to apply, the square root of the sum of the squares approach was used, since it is statistically more correct. This was considered to be important in mechanical design due to the wide variation in loading conditions, particularly the added significance of vertical accelerations.

Figure VII D-2 (page VII-24) illustrates the use of this procedure.

G. COMBINATION OF MAXIMUM EARTHQUAKE LOAD EFFECTS WITH EFFECTS OF OTHER LOADS

This subsection discusses the combinations of earthquake and dead, live and snow load effects.

The maximum earthquake load effects should be combined with the effects of other loads according to the following combinations:

For load combinations where addition produces the critical design load:

$$1.05\ Q_D + Q_L + Q_S + Q_E \text{ max} \qquad \text{(VII-24)}$$

For load combinations where the loads acting in opposite directions produce the critical design load, such as in overturning calculations,:

$$0.95\ Q_D - Q_E \text{ max (strength design)} \qquad \text{(VII-25)}$$

$$0.75\ Q_D - Q_E \text{ max (working stress design)} \qquad \text{(VII-26)}$$

where:

Q_D = dead load effect

Q_L = live load effect

Q_S = snow load effect

Q_E = earthquake load effect

The load combinations presented assume that vertical acceleration will be taken into account. Considering equation VII-24, the comparable equation in ATC 3-06 includes a multiplier of 1.2 rather than 1.05 for the dead load effect, Q_D. ATC 3-06 uses this 1.2 factor to account for vertical earthquake induced forces, i.e., increased dead load, but uses it as a constant that does not vary in different earthquake zones. These criteria include vertical earthquake induced forces as a portion, 0.6, of the horizontal force, both of which take into account varying seismic zones. Looking at equation VII-25, the comparable equation in ATC 3-06 includes a multiplier of 0.80 rather than 0.95 for the dead load effect. The increase from 0.80 to 0.95 is accounted for by the direct inclusion of vertical earthquake forces similar to the consideration for equation VII-24.

The multiplier for dead load resistance to overturning in equation VII-25 (strength design) is reduced from 0.95 to 0.75 in equation VII-26 (working stress design) to equate the results of calculations for dead and earthquake loads acting in opposite directions. In strength design, the loadings are those whose load effects are compared to the full expected strength of the member. In working stress design, the loadings are reduced to be compared to stresses containing a factor of safety. Dead loads do not have the uncertainty of other loadings. Thus, when load effects are subtractive, the factor of safety has to be accounted for. This effect is illustrated in Figure VII D-2 (page VII-24)

H. DESIGN EXAMPLES

Eight design examples have been included in this section to illustrate the application of the suggested criteria presented previously in this chapter. A cross section of examples was selected to include as many different types of applications as possible. In addition, a table showing maximum pipe spans for various types of pipe has been included as part of Figure VII D-11, Suspended Pipe Design Example. This table may be used as a reference when determining pipe support spacing.

CHLORINE TANK SCALE

ASSUME:

DEAD LOAD: EMPTY - $1600^{\#}$

FULL - $\underbrace{2000^{\#}}_{Cl_2} + \underbrace{1600^{\#}}_{\text{CYLINDER}} + \underbrace{300^{\#}}_{\text{SCALE}} = 3900^{\#}$

FROM MAP, $A_a = 0.30$

SYSTEM: CONTINUOUS OPERATION ESSENTIAL

CYLINDER IS FULL.

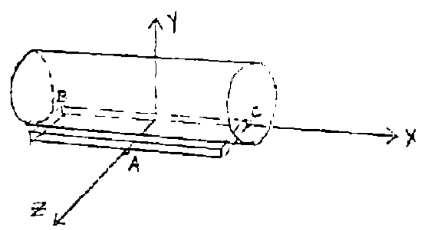
USE WORKING STRESS DESIGN (EQ. VII-5)

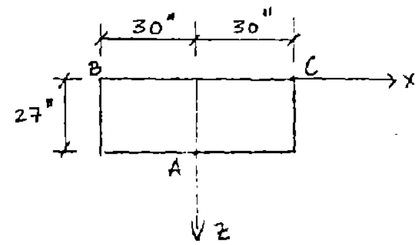
$$F_W = 0.6\ A_a\ C_w\ a_x\ W \qquad A_a = 0.30,\quad C_w = 2.4,\quad a_x = 1$$

ASSUME CYLINDER FULL.

$$F_W = 0.6\,(.30)(2.4)(3.9^K)$$

$$F_W = 1.685^K$$

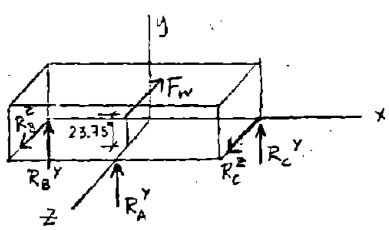




ASSUME A POINT VERTICAL SUPPORT AT 'A' (NO HORIZONTAL SHEAR). SUPPORTS AT 'B' AND 'C' ARE HINGED.

EARTHQUAKE EFFECTS

1) EQ IN -Z DIRECTION



$\Sigma F^Y = 0: \quad R_B^Y = -R_A^Y - R_C^Y$

$\Sigma F^Z = 0: \quad R_B^Z = F_W - R_C^Z$

$\Sigma M^X = 0: \quad R_A^Y(27") + F_W(23.75") = 0$

$R_A^Y = -.880\ F_W$

$\Sigma M^Z = 0: \quad R_C^Y(30") = R_B^Y(30") \quad \Rightarrow \quad R_C^Y = R_B^Y$

$\Sigma M^Y = 0: \quad R_B^Z = R_C^Z$

$R_A^Y = -.880\,F_W = -1.483^K$

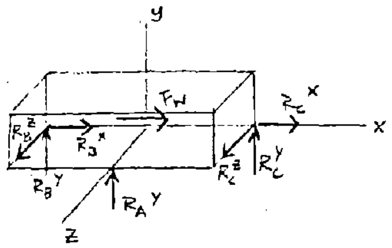
$R_B^Z = F_W - R_B^Z \quad \Rightarrow \quad R_B^Z = R_C^Z = \frac{1}{2} F_W = .842^K$

$R_B^Y = -R_A^Y - R_B^Y \quad \Rightarrow \quad R_B^Y = R_C^Y = -\frac{1}{2}(-1.483^K) = .742^K$

Figure VII D-6 Chlorine tank scale design example.

2) EQ IN +Z DIRECTION – REVERSE SIGNS FROM −Z CASE.

3) EQ IN +X DIRECTION



$\Sigma F^x = 0: \quad R_C^x + R_B^x = -F_w$

ASSUME INFINITELY RIGID BTWN. B & C.

$\therefore \quad R_C^x = R_B^x = -.5\,F_w = -.842^K$

$\Sigma F^y = 0: \quad R_A^y + R_B^y + R_C^y = 0$

$\Sigma F^z = 0: \quad R_B^z = -R_C^z$

$\Sigma M^x = 0: \quad R_A^y = 0$

$\Sigma M^z = 0: \quad R_B^y(30'') + F_w(23.75'') - R_C^y(30'') = 0 \Rightarrow R_C^y = R_B^y + .792\,F_w$

$\Sigma M^y = 0: \quad R_B^z(30'') + F_w(13.5'') - R_C^z(30'') = 0$

$R_B^z - R_C^z = -(13.5/30)F_w = -.758^K \quad$ BUT $\quad R_B^z = -R_C^z$

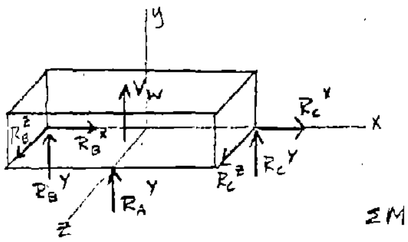
$R_B^z = -.379^K \qquad R_C^z = .379^K$

$R_B^y = -R_C^y \Rightarrow R_C^y = \tfrac{1}{2}(.792\,F_w) = .667^K$

$R_B^y = -.667^K$

4) EQ IN −X DIRECTION – REVERSE SIGNS FROM +X CASE

5) EQ IN +Y DIRECTION



$V_W = .6F_W = .6(1.685^K) = 1.011^K$

$\Sigma F^x = 0: \quad R_C^x = -R_B^x$

$\Sigma F^z = 0: \quad R_C^z = -R_B^z = 0$

$\Sigma F^y = 0: \quad V_W + R_A^y + R_C^y + R_B^y = 0$

$\Sigma M^y = 0: \quad R_B^z = R_C^z \quad$ BUT $\quad R_C^z = -R_B^z \quad \therefore R_B^z = R_C^z = 0$

$\Sigma M^z = 0: \quad R_C^y = R_B^y$

$\Sigma M^x = 0: \quad R_A^y(27'') = -(13.5'')V_W \Rightarrow R_A^y = -.506^K$

$R_C^y = R_B^y = \tfrac{1}{2}(-V_W - R_A^y) = -.253^K$

6) EQ IN −Y DIRECTION – REVERSE SIGNS FROM +Y CASE

MAXIMUM EARTHQUAKE EFFECT & COMBINATION WITH DEAD LOAD

DEAD LOAD = 3.9^K $\qquad \Sigma M^x = 0: \quad (3.9^K)(13.5'') = (R_A^y)_D\,(27'')$

$(R_A^y)_D = 1.95^K$

$(R_B^y)_D = (R_C^y)_D = .975^K$

Figure VII D-6 (cont.)

A) OVERTURNING AT 'A'

CRITICAL CASE $\begin{cases} +Y \text{ EQ} \\ -Z \text{ EQ} \end{cases}$

$$Q_{E\ MAX} = \sqrt{Q_x^2 + Q_y^2 + Q_z^2} \qquad (\text{EQ. VII-23})$$

$$(R_A{}^Y)_{E\ MAX} = \sqrt{(R_A{}^Y)_x^2 + (R_A{}^Y)_y^2 + (R_A{}^Y)_z^2} = \sqrt{(-.506^k)^2 + (-1.483^k)^2} = -1.567^k$$

EQ

$$0.75\,Q_D - Q_{E\ MAX} = 0.75(1.95^k) - 1.567^k = -.104^k \downarrow \quad (\text{EQ. VII-26})$$

B) MAXIMUM SHEAR AT 'B' AND 'C'

EQ IN X-DIRECTION: $R_B{}^z = .379^k$, $R_B{}^x = .842^k$

$$V_{MAX} = \sqrt{(.379^k)^2 + (.842^k)^2} = .923^k$$

EQ IN Z-DIRECTION: $R_B{}^z = .842^k$

$$V_{MAX} = .842^k$$

C) PULLOUT AT 'B' AND 'C'

CRITICAL CASE FOR R_B: $\begin{cases} +y \text{ EQ} \\ +x \text{ EQ} \\ +z \text{ EQ} \end{cases}$

$$(R_B{}^Y)_{E\ MAX} = \sqrt{(R_B{}^Y)_x^2 + (R_B{}^Y)_y^2 + (R_B{}^Y)_z^2}$$

$$= \sqrt{(.667^k)^2 + (.253^k)^2 + (.742^k)^2} = 1.029^k$$

$$0.75\,Q_D - Q_{E\ MAX} = 0.75\,(.975)^k - 1.029^k = -.298^k \downarrow \quad (\text{EQ. VII-26})$$

SUMMARY

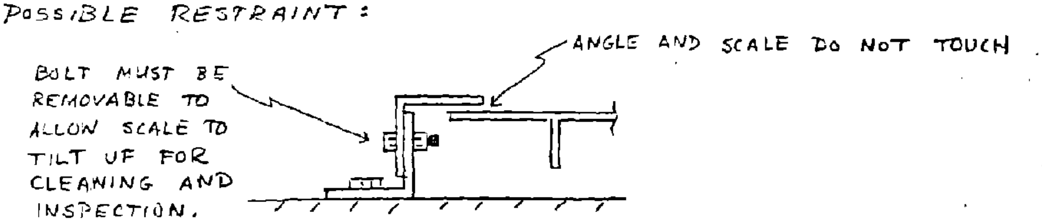
ANALYSIS FOR EMPTY TANK WOULD NOT BE CRITICAL.

ALONG EDGE CONTAINING PT. 'A', DESIGN RESTRAINT FOR $.104^k$ UPLIFT.

AT 'B' & 'C', DESIGN EACH CONNECTION FOR $.923^k$ SHEAR
$.298^k$ PULLOUT

USE SIMILAR ANALYSIS TO DETERMINE WHETHER STRAPPING TANK ONTO SCALE IS NECESSARY. FOR THIS EXAMPLE, IT IS ASSUMED THAT STRAPPING IS REQUIRED.

POSSIBLE RESTRAINT:



ALLOWABLE STRESSES MAY BE INCREASED BY 1/3 (AISC 1.5.6(204))

Figure VII D-6 (cont.)

WATER TANK ON SADDLE

ASSUME: 15,000 GALLON TANK

9'-0" DIAMETER AND 32'-0" AVERAGE LENGTH

FROM MAP, $A_a = 0.40$

ULTIMATE BEARING STRENGTH OF FOUNDATION SOIL = 6 KSF

CONTINUOUS OPERATION ESSENTIAL

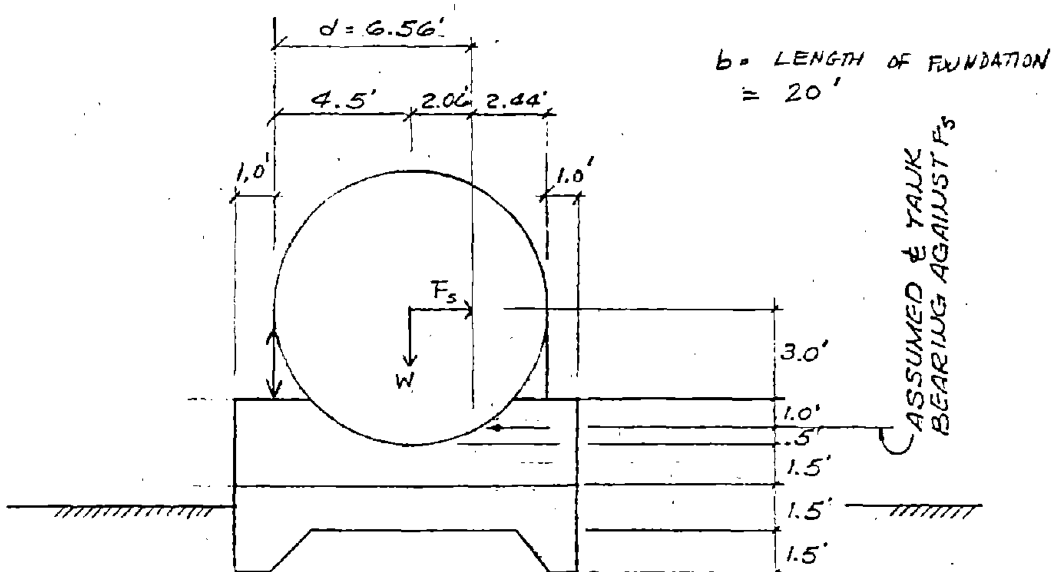
FOR DESIGN, ASSUME TANK IS FULLY FILLED.

$$\text{WEIGHT OF WATER} = \frac{62.4\,\pi D^2 \ell}{4 \times 1000}, \quad \text{WHERE } D = 9 \text{ FT}, \ \ell = 32 \text{ FT}.$$

$$= \frac{62.4\,\pi\, 9^2\, 32}{4000} = 127^{K}$$

$$\text{WEIGHT OF TANK} = 16^{K}$$

$$\text{WT. OF TANK + CONTENTS} = 143^{K}$$



USE STRENGTH DESIGN; (EQ. VII-2)

$F_S = 0.6\, A_a\, C_s\, a_x\, W$ IN WHICH $A_a = 0.4$, $C_s = 3$, $a_x = 1.0$, W = DEAD LOAD OF TANK AND CONTENTS

F_S (HORIZ.) $= (0.6)\, 0.4 \times 3.0 \times 1.0\, W = .72\, W$

V_S (VERT.) $= 0.6\,(1.2\,W) = .43\,W$

Figure VII D-7 Water tank on saddle design example

STRAP TENSION FROM OVERTURNING MOMENT:

Q_E (HORIZ.) $= .72W(4.0)/6.56 = .439W$

Q_E (VERT.) $= 0.43W/2 = .22W$

$Q_{E\,MAX} = W\sqrt{.439^2 + .22^2} = .49\,W$ (EQ. VII-23)

DESIGN $Q = .95Q_D - Q_{E\,MAX} = .95\frac{W}{2} - .49W = -.02W$ (TENSION)

$= .02 \times 143^K = 2.1^K$ OR 1.1^K EACH STRAP OF TWO

$A_{STRAP\ REQ'D} = \frac{1.1^K}{22.0 \times 1.7} = .03\ \square''$ ASSUME 1/4" PLATE W/ 2-3/4" ϕ SINGLE SHEAR

WIDTH REQ'D $= \frac{.03}{.25} = .11''$

NO. 3/4" ϕ BOLT $= \frac{1.1^K}{4.42 \times 1.6} = .16$

USE 1/4" x 4" STRAP & 2-3/4" ϕ A307 BOLTS - SINGLE SHEAR

SADDLE (2-12" WIDE SADDLES)

FOR REINFORCEMENT AT BOTTOM OF SADDLE,
MOMENT IN SADDLE WHEN STRAP HAS MAXIMUM TENSION:

PT. OF MAXIMUM MOMENT:

$1.1^K = (.150^K/FT^3)(4.5')(1')x$ WHERE x IS PT. OF MAX. MOMENT AS MEASURED FROM LEFT END

$x = 1.63'$

$M_{MAX} = (.150^K/ft^3)(4.5')(1')\frac{(1.63')^2}{2} - (1.1^K)(1.63') = .657'^K$

PROVIDE MINIMUM REINFORCEMENT AT BOTTOM OF SADDLE: 2-#5

RESISTANCE TO OVERTURNING (TRANSVERSE AXIS)

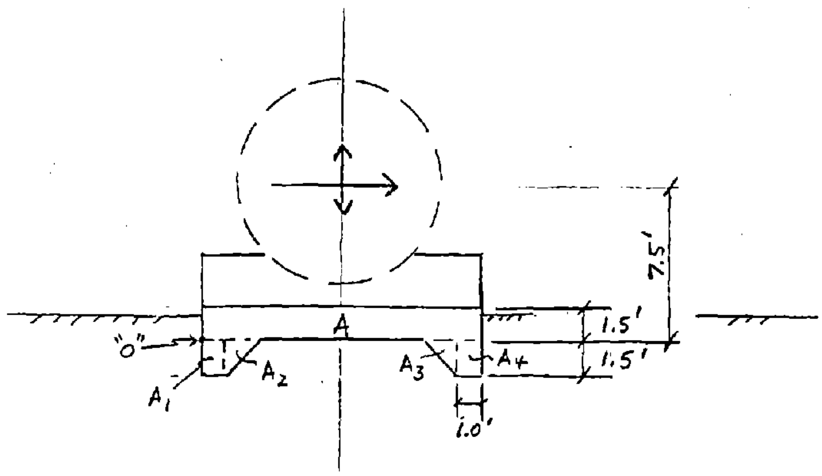


Figure VII D-7 (cont.)

ITEM	W (K)	MOM. HORIZ. FORCES ABOUT "O" ('K)	MOM. VERT. FORCES ABOUT "O" ('K)	DL MOMENT ABOUT "O" ('K)
WATER & TANK	143	$.72 \times 7.5W = 772$	$.43 \times 55W = 338$	$5.5W = 787$
SADDLES	10	$.72 \times 3.0W = 22$	$.43 \times 5.5W = 24$	$5.5W = 55$
FOOTING A	50	$.72 \times .75W = 27$	$.43 \times 5.5W = 118$	$5.5W = 275$
FOOTING A_1	5	$-.72 \times .75W = -3$	$.43 \times .5W = 1$	$0.5W = 3$
FOOTING A_2	3	$-.72 \times .50W = -1$	$.43 \times 1.5W = 2$	$1.5W = 5$
FOOTING A_3	3	$-.72 \times .50W = -1$	$.43 \times 9.5W = 12$	$9.5W = 29$
FOOTING A_4	5	$-.72 \times .75W = -3$	$.43 \times 10.5W = 23$	$10.5W = 53$
Σ	219	813	518	1207

TRANSVERSE $Q_{E\,MAX} = \sqrt{813^2 + 518^2} = 964'^{K}$ (EQ. VII-23)

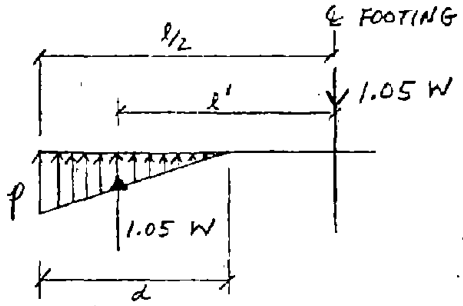
NET OVERTURNING $= 0.95 \times 1207 - 964 = 183'^{K}$ (NO NET UPLIFT) (EQ. VII-25)

(RESULTANT NOT REQ'D TO BE WITHIN MIDDLE THIRD OF BASE)

<u>SOIL MAX. PRESSURE</u>

$M_{E\,MAX} = 964'^{K}$

$W = 219^{K}$



$1.05\,(W) \times \ell' = M_E$

$\ell' = \frac{M_E}{1.05\,W} = \frac{964'^{K}}{1.05 \times 219^{K}} = 4.19'$

$\ell/2 - \ell' = \frac{1}{3} d$

$d = 3\,(\ell/2 - \ell') = 3\,(5.5' - 4.19') = 3.92'$

$p = \left(\frac{1.05\,W}{d \times b}\right) \times 2 = \left(\frac{1.05 \times 219^{k}}{(3.92')(20')}\right) \times 2 = 5.87$ KSF < 6 KSF ✓OK

SLAB SHALL BE DESIGNED AS LONGITUDINAL BEAM BETWEEN THICKENED EDGES WHICH SPAN BETWEEN SADDLES, USING THE LOADS AND PRESSURES DETERMINED ABOVE.

Figure VII D-7 (cont.)

END-MOUNTED EQUIPMENT (CHEMICAL FEEDER)

ASSUME:

WEIGHT OF EQUIPMENT AND CHEMICALS:

$W_1 = 1.0^k$

$W_2 = 1.2^k$

$W_3 = .3^k$

FROM MAP, $A_a = 0.40$

SYSTEM: SHUT DOWN OF 2 WEEKS IS ACCEPTABLE.

USE WORKING STRESS METHOD.

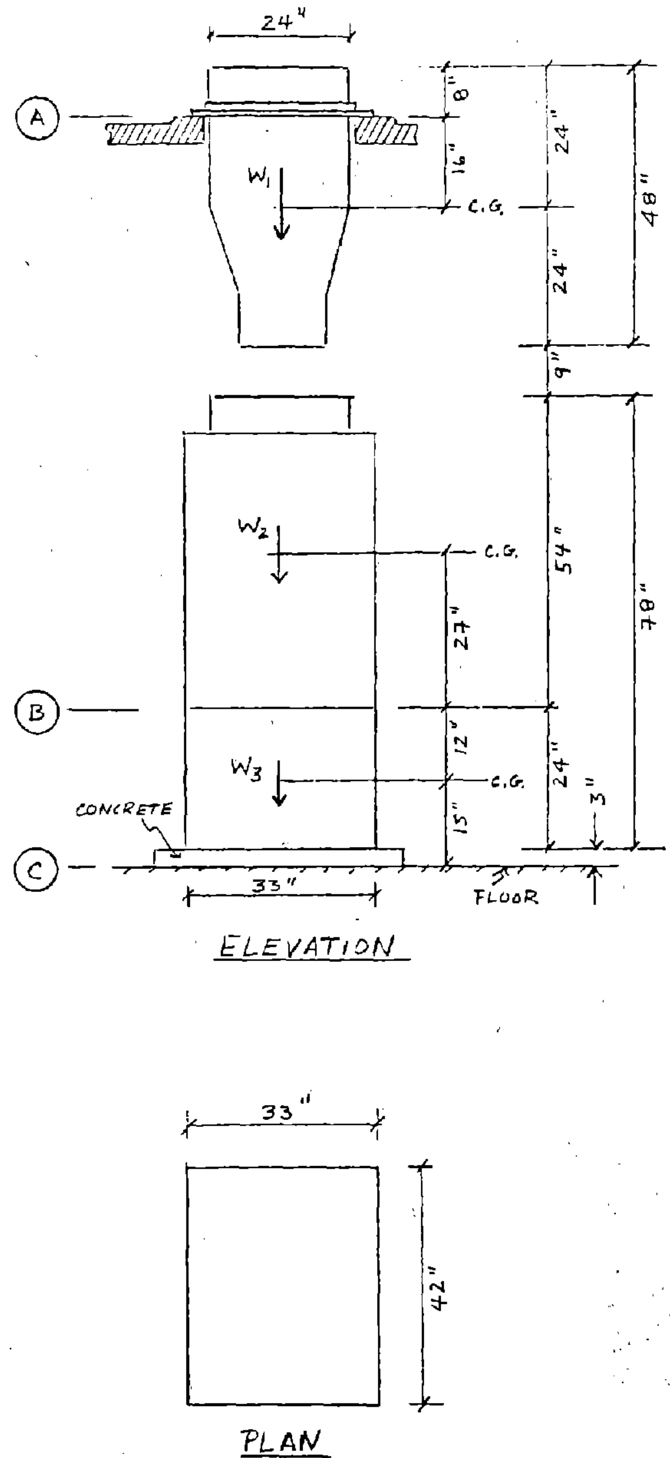
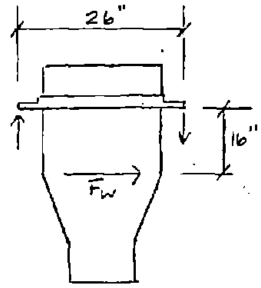


Figure VII D-8 End-mounted equipment (chemical feeder) design example

REACTIONS AT (A)



$F_W = 0.6\, A_a\, C_w\, a_x\, W$ (EQ. VII-5)
$= 0.6(0.4)(1.6)(2)(1^K)$

$A_a = 0.4$
$C_W = 1.6$
$a_x = 1 + \frac{130}{130} = 2$
$W_1 = 1^K$

$F_W = .768^K$

VERTICAL FORCE $V_W = .6 F_W$

$V_W = .6(.768^K) = .461^K$

PERIMETER OF CIRCLE WITH $D = 26''$ $= \pi d = \pi(26'') = 81.68''$

I OF CIRCLE $= \pi r^3 = \pi(13'')^3 = 6902\ in^3$

SECTION MODULUS $S = 6902\ in^3 / 13'' = 531\ in^2$ FOR 1" WIDTH.

$Q_D = \frac{1^K}{\text{PERIMETER}} = \frac{1^K}{81.68''} = .0122\ ^K/_{''}$

$M = F_W(16'') = (.768^K)(16'') = 12.29''^K$

$f_c = \frac{M}{S} = \frac{12.29''^K}{531\ in^2} = .023\ ^K/_{''}$ MAX. BENDING STRESS.

$Q_x = Q_y = .023\ ^K/_{''}$

$Q_Z = \frac{V_W}{\text{PERIMETER}} = \frac{.461^K}{81.68''} = .006\ ^K/_{''}$

$Q_{E\ MAX} = \sqrt{2(.023\ ^K/_{''})^2 + (.006\ ^K/_{''})^2} = .033\ ^K/_{''}$ (EQ. VII-23)

$Q_D = .0122\ ^K/_{''}$

UPLIFT: $Q = .75\, Q_D - Q_{E\ MAX} = .75(.0122\ ^K/_{''}) - .033\ ^K/_{''} = -.024\ ^K/_{''}$ (EQ. VII-26)

SHEAR: $V_x = V_y = \frac{F_W}{\text{PERIMETER}} = \frac{.768^K}{81.68''} = .0094\ ^K/_{''}$ $V_Z = 0$

$(V_E)_{MAX} = \sqrt{2(.0094\ ^K/_{''})^2} = .0133\ ^K/_{''}$ NO SHEAR FROM DL. (EQ. VII-23)

BASE BOLT SPACING ON THESE VALUES: UPLIFT: $.024\ ^K/_{''}$
SHEAR: $.013\ ^K/_{''}$

REACTIONS AT (B)

$F_W = 0.6\, A_a\, C_w\, a_x\, W$ (EQ. VII-5)
$= 0.6(0.40)(1.6)(1.2^K)$

$A_a = 0.40$
$C_W = 1.6$
$a_x = 1$ (ASSUMING GROUND FLOOR)
$W_2 = 1.2^K$

$F_W = .461^K$

VERTICAL FORCE - $V_W = .6 F_W$

$V_W = .6(.461^K) = .276^K$

ASSUME PINNED CONNECTIONS AT 4 CORNERS.

1) DEAD LOAD:

$Q_D = \frac{1}{4}(W_2) = \frac{1}{4}(1.2^K) = .3^K$ EACH CORNER

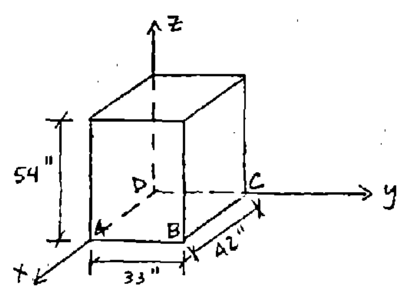
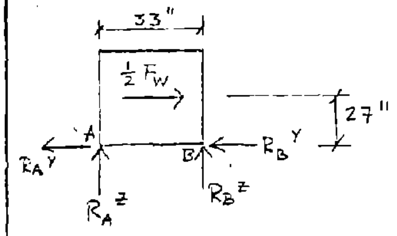


Figure VII D-8 (cont.)

2) EQ IN Y-DIRECTION : $\frac{1}{2} F_W = .461^k/2 = .231^k$



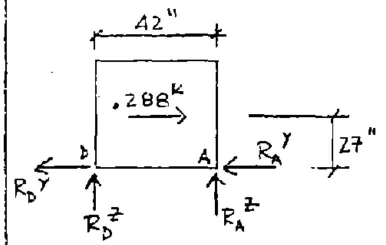
For Tension or Compression :

$$-R_A{}^z = R_B{}^z = \frac{1}{33''}(.231^k)(27'') = .189^k = Q_Y$$

$$-R_D{}^z = R_C{}^z = .189^k = Q_Y$$

For Shear : $Q_Y = R_B{}^Y = R_A{}^Y = \frac{.231^k}{2} = .116^k$

3) EQ. IN X-DIRECTION :



For Tension or Compression :

$$Q_x = R_A{}^z = -R_D{}^z = R_B{}^z = -R_C{}^z = \frac{1}{42''}(.231^k)(27'')$$

$$Q_x = .149$$

For Shear : $Q_x = \frac{1}{2}(.231^k) = .116^k$

4) EQ IN Z-DIRECTION :

For Tension or Compression : $Q_z = R_A{}^z = R_B{}^z = R_C{}^z = R_D{}^z = \frac{1}{4} V_W = \frac{1}{4}(.276^k) = .069^k$

No shear.

FOR TENSION OR COMPRESSION : VERTICAL $Q_{E\,MAX} = \sqrt{Q_x{}^2 + Q_y{}^2 + Q_z{}^2}$

$$= \sqrt{(.149)^2 + (.189)^2 + (.069)^2}$$

$$= .250^k \qquad \text{(EQ. VII-23)}$$

FOR SHEAR : HORIZONTAL $Q_{E\,MAX} = \sqrt{(.116)^2 + (.116)^2 + 0} = .164^k$

TENSION CORNERS : $Q = .75\,Q_D - Q_{E\,MAX} = .75(.3^k) - .250^k = -.025^k$ (EQ. VII-26)

COMPRESSION CORNERS : $Q = 1.05\,Q_D + Q_{E\,MAX} = 1.05(.3^k) + .250^k = .565^k$ (EQ. VII-24)

CONNECTIONS BETWEEN UNITS AT B ARE EACH REQUIRED TO CARRY :

$.025^k$ TENSION
$.565^k$ COMPRESSION
$.164^k$ SHEAR

REACTIONS AT C

$F_W = 0.6\, A_a\, C_W\, a_x\, W$ (EQ. VII-5) $\qquad A_a = 0.40 \qquad a_x = 1$

$C_W = 1.6 \qquad W_2 = 1.2^k \qquad W_3 = 0.3^k$

$F_W{}^2 = 0.6(0.4)(1.6)(1)(1.2) = .461^k$

$F_W{}^3 = 0.6(0.4)(1.6)(1)(0.3) = .115^k$

$V_W = 0.6(F_W{}^2 + F_W{}^3) = 0.6(.461^k + .115^k) = .346^k$

1) DEAD LOAD : $Q_D = \frac{1}{4}(1.2^k + 0.3^k) = .375^k$ AT EACH CORNER

Figure VII D-8 (cont.)

2) EQ IN Y-DIRECTION:

VERTICAL $Q_y = -R_A{}^z = R_B{}^z = -R_D{}^z = R_C{}^z = \frac{1}{33''}\left[\frac{.461}{2}(27''+24'') + \frac{.115}{2}(12'')\right]$
$= .377^K$

HORIZ $Q_y = (.461^K + .115^K)\frac{1}{4} = .144^K$

3) EQ IN X-DIRECTION:

VERTICAL $Q_x = \frac{1'}{42''}\left[\frac{.461^K}{2}(27''+24'') + \frac{.115}{2}(12'')\right] = .296^K$

HORIZ. $Q_y = .144^K$

4) EQ. IN Z-DIRECTION:

VERTICAL $Q_z = \frac{1}{4}(.346^K) = .087^K$

HORIZ. $Q_z = 0.$

VERTICAL $Q_{E\ MAX} = \sqrt{Q_x{}^2 + Q_y{}^2 + Q_z{}^2}$ (EQ. VII-23)

$= \sqrt{(.296^K)^2 + (.377^K)^2 + (.087^K)^2} = .487^K$

$Q = .75\,Q_D - Q_{E\ MAX} = .75(.375^K) - .487^K = -.206^K$ (EQ. VII-26)

HORIZONTAL $Q_{E\ MAX} = \sqrt{(.144)^2 + (.144)^2} = .203^K$ NO DEAD LOAD CONTRIBUTION TO SHEAR.

EACH CONNECTION AT C MUST CARRY $.206^K$ UPLIFT
$.203^K$ SHEAR.

ALLOWABLE STRESSES MAY BE INCREASED BY 1/3 (AISC 1.5.6 (209))

Figure VII D-8 (cont.)

ELEVATED WATER TANK

ASSUME:

75,000 GALLON TANK

18 FT. DIAMETER:

AVERAGE HT. OF TANK:

$$\pi r^2 \ell = \frac{75{,}000 \text{ GAL}}{7.48 \text{ GAL/FT}^3}$$

$$\ell = \frac{10027 \text{ FT}^3}{\pi (9')^2}$$

$\ell = 39.4'$, SAY 40 FT.

TOWER PLAN DIMENSIONS:
20'×20' TOP
40'×40' BOTTOM

FROM MAP, $A_a = A_v = 0.40$

SOIL TYPE 2

TWO WEEK SHUTDOWN OK

TRUE VERTICAL DIMENSIONS (NOT IN PLANE OF SIDE) GIVEN ON DIMENSION LINE

MEMBER SLOPE DIMENSIONS GIVEN (   ).

APPLY 1^K LOAD ON TOWER AS SHOWN. RESULTING FORCES IN EACH MEMBER ARE SHOWN ▭.

OFFSET MASS WITH 5% ECCENTRICITY AND APPLY 1^K LOAD. RESULTING FORCES IN EACH MEMBER ARE SHOWN ▱.

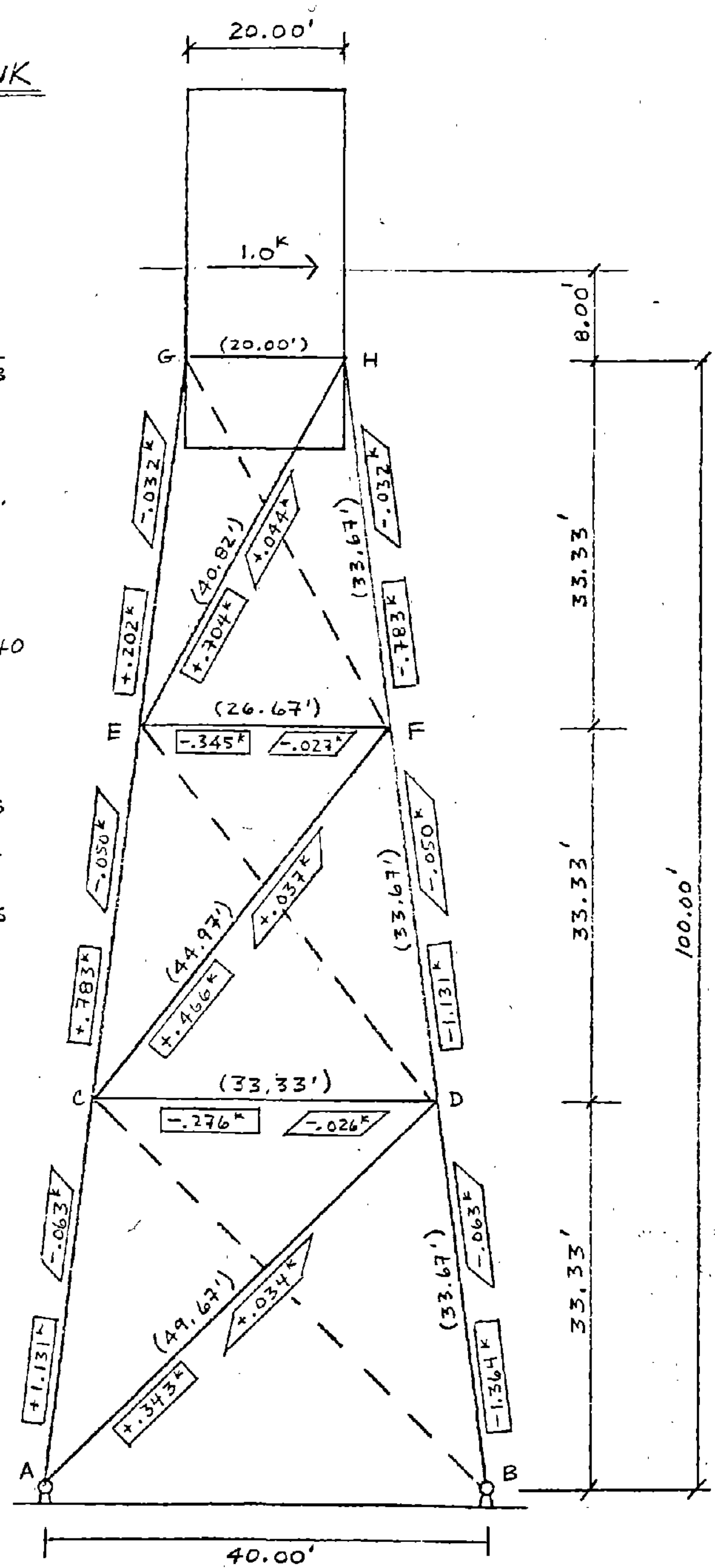


Figure VII D-9 Elevated water tank design example

PRELIMINARY SIZING AND PERIOD

MEMBER	L (FT)	EST. REQ'D AREA (IN²)	EST. SECTION	AREA (IN²)	WT. (LB/FT)	u (K)	u^2L/A $\left(\frac{K^2-FT}{IN^2}\right)$	n = NO. OF MEMBERS SAME SIZE	WT = n(WT.)L (K)
AB	40.0	∞							
AD	49.67	5.5	⌐⌐ 6×4×5/16	6.05	20.6	+.343	.9659	8	8.2
AC	33.67	40.0	W14×145	42.7	145	+1.131	1.0086	4	19.5
BD	33.67	40.0	W14×145	42.7	145	-1.364	1.4670		
CD	33.33	8.0	⌐⌐ 8×8×1/2	13.5	46.0	-.276	.1881	4	6.1
CF	44.97	4.0	⌐⌐ 5×3½×1/4	4.13	14.0	+.466	2.3645	8	5.0
CE	33.67	37.0	W14×132	38.8	132	+.783	.5320	4	17.8
DF	33.67	37.0	W14×132	38.8	132	-1.131	1.1100		
EF	26.67	8.0	⌐⌐ 6×6×3/8	8.72	29.8	-.345	.3640	4	3.2
EH	40.82	6.0	⌐⌐ 6×3½×5/16	5.74	19.6	+.704	3.5246	8	6.4
EG	33.67	35.0	W12×120	35.3	120	+.202	.0389	4	16.2
FH	33.67	35.0	W12×120	35.3	120	-.793	.5848		
GH	20.00	∞							
Σ							12.148		82.4

TOWER WT. = $W(1+.05$ FOR CONN.$) = 82.4^k(1.05) = 86.5^k$

$\frac{1}{2}$ TOWER WT. = 43^k

WT. OF TANK = 66^k (ASSUMING ½" PL)

WT. OF WATER = $(62.4\ lb/ft^3)(75000\ GAL)\left(\frac{ft^3}{7.48\ GAL}\right) = 626^k$

$\Bigg\}\ W = 735^k$

BY VIRTUAL WORK:

$(.5^k)\,\Delta = \Sigma \frac{u^2L}{AE}$ WHERE Δ = LATERAL DEFLECTION OF TANK DUE TO 1^k LOAD.

$$\Delta = \frac{1}{.5^k}\,\frac{12.149\ K^2\text{-}FT/IN^2}{29\times10^3\ KSI}\,(12\ IN/FT)$$

$$\Delta = .01005''$$

$T = 2\pi\sqrt{m/\lambda}$ WHERE $\lambda = \frac{F}{\Delta}$, $m = \frac{W}{g}$

$$T = 2\pi\sqrt{\frac{(735^k)(.01005'')}{(32.2\ '/s^2)(1.0^k)(12''/')}} = .869\ sec$$

Figure VII D-9 (cont.)

FROM EQ. VII-17 USING WORKING STRESS METHOD:

$$F_W^E = \frac{0.6 A_v S C_w W}{R T^{2/3}} \qquad A_v = 0.40$$

$$C_w = 1.6$$

$$= \frac{(.6)(.4)(1.2)(1.6)(735^k)}{2(.869)^{2/3}} \qquad S = 1.2, \quad R = 2, \quad W = 735^k$$

$$F_W^E = 186^k \qquad T = .869 \text{ sec}$$

DEFLECTION: $\left(\frac{186^k}{2}\right)\Delta = \sum \frac{(186\,u)^2 L}{AE}$

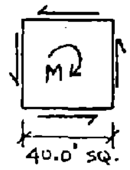
$$\Delta = \frac{2}{186}(186)^2 \sum \frac{u^2 L}{AE} = 2(186)\frac{12.433}{29 \times 10^3}(12) = 1.91''$$

DIRECT LOAD STRESS $= (186)u$

STRESS DUE TO MASS OFFSET WITH 5% ECCENTRICITY:

$M = (.05)(40.0')(1^k) = 2'^k$

SHEAR ON EACH SIDE $= \frac{2'^k}{4(20')} = .025^k$



u' = BAR FORCE DUE TO 1^k LOAD

ECCEN. LOAD STRESS $= (186)u'$

Q_D = FORCE DUE TO DL OF WATER, TANK, TOWER SELF-WT.

FOR TOP LEVEL OF COLUMNS: $Q_D \simeq \frac{1}{4}\left[66^k + 626^k + \underbrace{\tfrac{1}{3}(86.5^k)}_{\frac{1}{3}\text{ TOWER WT.}}\right] = 180^k$

FOR MIDDLE LEVEL: $Q_D \simeq \frac{1}{4}\left[66^k + 626^k + \tfrac{2}{3}(86.5^k)\right] = 187^k$

FOR LOWER LEVEL: $Q_D \simeq \frac{1}{4}\left[66^k + 626^k + 86.5^k\right] = 194^k$

FROM EQ. VII-23:

$$(Q_E)_{MAX} = \sqrt{(Q_x)^2 + (Q_y)^2 + (Q_z)^2}$$

Q_z = FORCE IN MEMBER WITH EQ APPLIED VERTICALLY

FROM SEC. 2a,

$V_W = 0.6\,F_W = 0.6(0.6)A_a C_N a_x W$ WHERE $A_a = 0.4$, $C_W = 1.6$, $a_x = 1$

$= (0.6)(0.6)(0.4)(1.6)(1)(778^k)$ $\quad W = 778$ (LOWER COL. LEVEL)

$= 179^k$ (LOWER LEVEL) $\quad = 750$ (MIDDLE LEVEL) $\quad = 721$ (UPPER LEVEL)

FOR COLUMNS, $Q_z = \frac{1}{4}(179^k) = 44.8^k$ (LOWER LEVEL)

FOR WEB MEMBERS, $Q_z = 0$

ASSUMING V_W APPLIED AT C.G. EFFECT OF OFFSET NOT CONSIDERED FOR VERTICAL EQ

FOR X & Y EARTHQUAKES:

COLUMNS: Q_x = (DIRECT LOAD STRESS) + (ECCEN. LOAD STRESS)

$Q_y = Q_x$

WEB MEMBERS: Q_x = (DIRECT LOAD STRESS) + (ECCEN. LOAD STRESS)

Q_y = ECCEN. LOAD STRESS

PLAN BRACING IN PLANE OF HORIZONTAL FRAMING FOR SECONDARY TORSIONAL EFFECTS HAS NOT BEEN ANALYZED.

Figure VII D-9 (cont.)

MEMBER	AREA (in^2)	L (FT)	u (K)	u' (K)	DIRECT LOAD STRESS (K)	ECCEN. LOAD STRESS (K)	Q_x (K)	Q_y (K)	Q_z (K)	Q_D (K)	$(Q_E)_{MAX}$	Q (K): $0.75Q_L - (Q_E)_{MAX}$	Q (K): $1.05Q_D + (Q_E)_{MAX}$	Q/A (ksi) ①	KL/r ②	ALLOWABLE STRESS F_a ③ COMPR. (ksi)	ALLOWABLE STRESS F_a ③ TENSION (ksi)	$\lvert Q/A \rvert < \lvert F_a \rvert$?
AD	6.05	49.67	.343	.034	63.8	6.3	70.1	6.3	0	0	70.4	70.4		11.63	278 / 218		29.3	OK
AC	42.7	33.67	1.131	-.063	210.3	-11.8	198.5	198.5	44.8	-194	284.3	138.8		3.25	102		29.3	OK
BD	42.7	33.67	-1.364	-.063	-253.7	-11.8	-265.5	-265.5	44.8	-194	-378.1		-581.8	-13.63	102	-16.92		OK
CD	13.5	33.33	-.276	-.026	-51.4	-56.2	-107.6	-56.2	0	0	-121.4		-121.4	-8.99	172	-9.07		OK
CF	4.13	44.97	.466	.037	86.7	6.8	93.5	6.8	0	0	93.7	93.7		22.70	288 / 240		29.3	OK
CE	38.8	33.67	.783	-.050	145.7	-9.4	136.3	136.3	43.3	-187	197.6	57.3		1.48	108		29.3	OK
DF	38.8	33.67	-1.131	-.050	-210.3	-9.4	-219.7	-219.7	43.3	-187	-313.7		-510.1	-13.15	108	-15.88		OK
EF	8.72	26.67	-.345	-.027	-64.1	-5.0	-69.1	-5.0	0	0	-69.3		-69.3	-7.95	170	-9.16		OK
EH	5.74	40.82	.704	.044	130.9	8.1	139.0	8.1	0	0	139.3	139.3		24.26	281 / 175		29.3	OK
EG	35.3	33.67	.202	-.032	37.6	-5.9	31.7	31.7	41.5	-180	61.1			NOT CRITICAL				
FH	35.3	33.67	-.783	-.032	-145.7	-5.9	-151.6	-151.6	41.5	-180	-218.4		-407.4	-11.54	129	-11.93		OK

① CRITICAL STRESS FOR LISTED MEMBER AND ITS SYMMETRICAL MEMBER, BASED ON GOVERNING LOAD COMBINATION: $0.75\,Q_D - (Q_E)_{MAX}$ (EQ. VII-26)

OR $1.05\,Q_D + Q_L + Q_S + (Q_E)_{MAX}$ (EQ. VII-24) ASSUMING $Q_L = Q_S = 0$

LOAD CONDITION WITHOUT EARTHQUAKE NEVER GOVERNS DESIGN.

$A = A_{GROSS}$ THIS IS UNCONSERVATIVE (~15 %) FOR TENSION MEMBERS. IF CRITICAL, BASE DESIGN ON NET SECTION.

② MAXIMUM KL/r : FOR COLUMNS: 200 (AISC 1.8.4(204))
FOR HORIZ. BRACING: 200
FOR DIAG. BRACING: 300

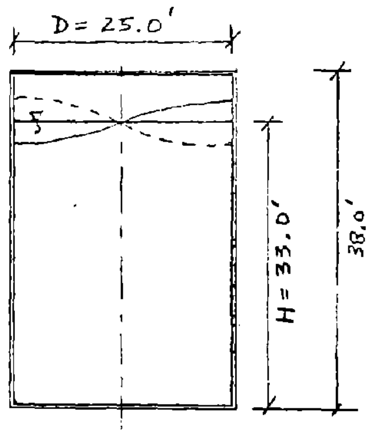
DIAGONAL BRACES: BASED ON r_{x-x}, IN-PLANE BENDING, ASSUMING CROSS BRACE ACTS AS RESTRAINT.
BASED ON r_{y-y}, OUT-OF-PLANE BENDING, ASSUMING CROSS BRACE NOT EFFECTIVE AS RESTRAINT.

③ ALLOWABLE STRESSES INCREASED BY 1/3 (AISC 1.5.6 (204))

Figure VII D-9 (cont.)

TANK ON GROUND

ASSUME:



FLAT BOTTOM STEEL TANK, COVERED, ANCHORED
25'-0" INSIDE DIAMETER
FROM MAP, $A_a = A_v = 0.40$
SHELL THICKNESS = 3/8"
ROOF: 5/16" THICK, WITH STIFFENERS
SOIL - STIFF CLAY
TWO-WEEK SHUTDOWN ACCEPTABLE
ALLOWABLE BEARING STRESS FOR FOUNDATION SOIL = 5 KSF
USE WORKING STRESS METHOD

$W_t = \pi (25.0')(38.0')(15.3 \text{ lb/ft}^2) = 46^k$

$W_r = \pi (12.5')^2 (25.0 \text{ lb/ft}^2) = 12^k$

$W_T = \frac{\pi D^2}{4} H \; (62.4 \text{ lb/ft}^3)$ $\quad D = 25.0', \; H = 33.0'$

$= 1011^K$

$\frac{D}{H} = \frac{25}{33} = .76$ FROM FIG. VII-5, $\frac{W_1}{W_T} = .85$, $\frac{W_2}{W_T} = .185$

$W_1 = .85\,(1011^k) = 859^k$

$W_2 = .185\,(1011^k) = 187^k$

HORIZONTAL SHEAR AT BASE OF TANK SHELL:

$$F_W^{CT} = \frac{1.2}{R} C_W \left[A_a \left(W_t + W_r + W_1^{CT} \right) + \frac{C_1}{2} W_2^{CT} \right] \quad \text{(EQ. VII-7)}$$

$A_a = 0.4$ (FIGURE VII-1)
$C_W = 1.6$ (TABLE VII-2)
$R = 2.0$ (PG. VII-31)

$C_1 = \frac{A_V S}{T^{2/3}}$ WHERE: $S = 1.2$ (PG. VII-31)
$A_V = 0.40$ (ATC 3-06, TABLE 1-B)
$T = K_p D^{1/2}$
$K_p = 0.58$ (FIG VII-7)
$T = (.58)(25.0)^{1/2} = 2.9$ SEC

$= \frac{.40\,(1.2)}{(2.9)^{2/3}}$

$= .236$

$$F_W^{CT} = \frac{1.2}{2.0}(1.6)\left[0.40\,(46^K + 12^K + 859^K) + \frac{.236}{2}(187^K)\right]$$

$F_W^{CT} = 373.3^K$

Figure VII D-10 Tank on ground design example

BENDING MOMENT JUST ABOVE BOTTOM OF TANK SHELL:

$$M_w^{CT} = \frac{1.2}{R} C_W \left[A_\alpha (W_t X_t + W_r H_r + W_1^{CT} X_1^{EBP}) + \frac{C_1}{2} W_2^{CT} X_2^{EBP}\right] \quad \text{(EQ. VII-9)}$$

$X_t = 19.0'$

$H_r = 38.0'$

FOR $D/H = .76$, FROM FIG. III-2, $\frac{X_1^{EBP}}{H} = .44$, $\frac{X_2^{EBP}}{H} = .80$

$X_1^{EBP} = (.44)(33.0') = 14.52'$

$X_2^{EBP} = (.80)(33.0') = 26.4'$

$$M_w^{CT} = \frac{1.2}{2.0}(1.6)\left\{0.4\left[(146^k)(19.0') + (12^k)(38.0') + (859^k)(14.52')\right] + \frac{.236}{2}(187^k)(26.4')\right\}$$

$M_w^{CT} = 5859.5\ ^{'k}$

BASE RING FOR ANCHOR BOLTS

ASSUME DISTANCE FROM OUTSIDE OF SHELL TO BOLT ℄ = 2.5"

BOLT RING DIAMETER $= \{25' + [2(3/8") + 2(2.5")]\frac{1}{12}\} = 25.48'$

BOLT RING CIRCUMFERENCE $= \pi d = 80.05'$

MAX. STRESS ON BOLTS DUE TO BENDING MOMENT:

$f_b = \frac{M}{S}$ $\qquad S = \pi r^2 = \pi(12.74')^2 = 509.9\ ft^2$

$f_b = \frac{5859.5^{'k}}{509.9\ ft^2} = 11.49\ ^{k/'}$

ASSUME ASTM A307 BOLTS, A36, $1\frac{1}{2}"\phi$: TENSILE CAPACITY = 30.92^k

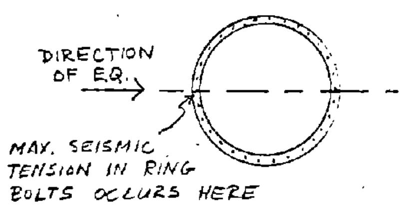
SPACING $= 30.92^k / 11.49^{k/'} = 2.69'$

NO. OF BOLTS $= \frac{80.05'}{2.69'} = 29.8$

(BOLTS DO NOT CARRY SHEAR; SHEAR CARRIED BY FRICTION AND BY BEARING ON FOUNDATION)

USE 30 1-½" φ A307 BOLTS

ASSUME CONTINUOUS BASE RING, WITH STIFFENER PLATES ON BOTH SIDES OF EACH BOLT.



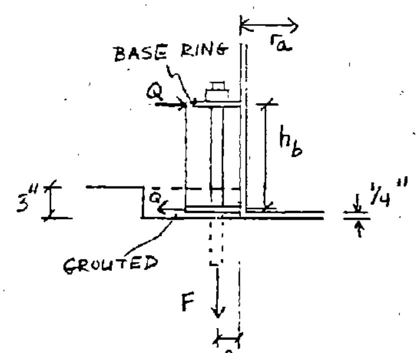


Figure VII D-10 (cont.)

FIND MAX. MOMENT IN RING DUE TO BOLT ECCENTRICITY :

$M_a = CQr_a$ (STRUCTURAL ENGINEERING HANDBOOK, GAYLORD & GAYLORD, 1968, SECS. 26-6, -8 (205))

$Q = \frac{Fe}{h_b}$ ASSUME $e = 2.5''$, $h_b = 12''$

F = ALLOWABLE LOAD ON BOLT = 30.92^k

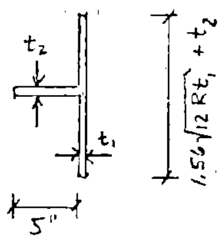
$Q = \frac{(30.92^k)(2.5'')}{12''} = 6.44^k$

$r_a = 12.5' + (3/8'')\frac{1}{12} = 12.53'$

$C \simeq .45$ for 30 BOLTS

$M_a = (.45)(6.44^k)(12.53') = 36.31'^k$

T-SECTION EFFECTIVE IN RESISTING M_a : (GAYLORD & GAYLORD (205))



$R = 12.5'$

$t_1 = 3/8''$

$t_2 = 1/2''$ (ASSUMED)

$1.56\sqrt{12Rt_1} + t_2 = 12.2''$, SAY $12''$

SO h_b ASSUMPTION OK.

AREA = $3/8''(12'') + (1/2'')(5'') = 7\ in^2$

CIRCUMFERENTIAL STRESSES ON T-SECTION;

1. TENSILE STRESS (ASSUME UNIFORM) DUE TO HYDROSTATIC PRESSURE
2. TENSILE STRESS (MAX. AT OUTER EDGE) DUE TO BOLT ECCENTRICITY
3. COMPRESSIVE STRESS DUE TO DYNAMIC FORCE OF WATER — SINCE THIS STRESS IS SUBTRACTIVE AND MINOR, NEGLECT.

SLOSHING OF WATER DOES NOT AFFECT THIS SECTION DIRECTLY — ITS EFFECT IS INCLUDED IN 2.

WELDING OF TANK SHELL TO VARIOUS COMPONENTS (I.E. RING, STIFFENER PLATES, SHEAR RESISTANCE AT BOTTOM) SHOULD BE BASED ON FORCES DETERMINED IN THIS ANALYSIS AND ON STRESSES PERMITTED IN AWSD1.1-80.

1 TENSILE STRESS DUE TO HYDROSTATIC PRESSURE

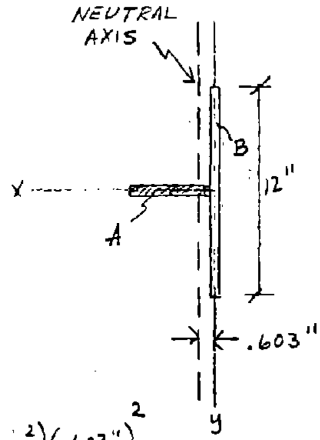
$$f_1 = \frac{pr(1')}{AREA} = \frac{(62.4\ lb/ft^3)(33.0')(12.5')(1')}{7\ in^2} = 3.68\ ksi$$

2 TENSILE STRESS DUE TO BOLT ECCENTRICITY

$$f_2 = \frac{M}{S} = \frac{36.31'^k}{S}$$

Figure VII D-10 (cont.)

SECTION	AREA (IN²)	$\overline{x}$ (IN)	$\overline{x}A$ (in³)
A	2.5	1.688	4.22
B	4.5	0	0
	$\Sigma A = 7.0$		$\Sigma\overline{x}A = 4.22$



$$\overline{x} = \frac{1}{\Sigma A}\Sigma\overline{x}A = .603''$$

$$I = I_A + A_A d_A^2 + I_B + A_B d_B^2$$

$$d_A = (2.5'' + \tfrac{3}{16}'') - .603'' = 2.084''$$

$$I = \frac{(\frac{1}{2}'')(5'')^3}{12} + (2.5\text{ in}^2)(2.084'')^2 + \frac{(12'')(\frac{3}{8}'')^3}{12} + (4.5\text{ in}^2)(.603'')^2$$

$$I = 17.755\text{ in}^4$$

AT OUTER EDGE, $S = \dfrac{I}{(5'' + \frac{3}{16}'') - .603''} = \dfrac{17.755\text{ in}^4}{4.585''} = 3.873\text{ in}^3$

$$f_{②} = \frac{M}{S} = \frac{36.31^k}{3.873\text{ in}^3} = 9.376\text{ ksi}$$

$$f_{①} + f_{②} = 3.68\text{ ksi} + 9.38\text{ ksi} = 13.06\text{ ksi} < 22\text{ ksi} \quad \checkmark\text{OK.}$$

FOOTING

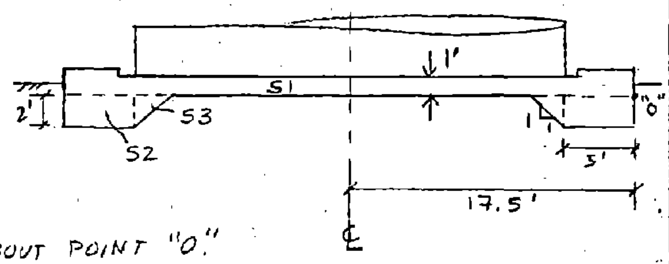
FOOTING MUST HAVE SUFFICIENT MASS SO THAT OVERTURNING DOES NOT OCCUR.

OVERTURNING MOMENT AT BASE OF STRUCTURE (HORIZONTAL E.Q.):

$$M_w = M_w^T + F_w^T h_x + \Sigma F_w' h_n \quad \text{(EQ. VII-16)}$$

ASSUME DIMENSIONS OF FOOTING AS SHOWN:

SECTION	VOL. (FT³)	WT (K)
S1	572.6	85.9
S2	942.0	141.3
S3	354.0	53.1
		$W_{FTG} = 280.3$



MOMENTS ARE EVALUATED ABOUT POINT "O."

$$M_w^T = \frac{1.2}{R} C_W \left[A_a (W_t X_t + W_r H_r + W_1^T X_1^{IBP}) + \frac{C_1}{2} W_2^T X_2^{IBP}\right]$$

FROM FIG. VII-8, FOR $D/H = .76$, $X_1^{IBP}/H = .6$

$X_2^{IBP}/H = .7$

$$X_1^{IBP} = .6\,(33') = 19.8'$$

$$X_2^{IBP} = .7\,(33') = 23.1'$$

Figure VII D-10 (cont.)

$$M_W^T = \frac{1.2}{2.0}(1.6)\Big\{0.4\big[(46^k)(19.0') + (12^k)(38.0') + (859^k)(19.8')\big] + \frac{.236}{2}(187^k)(23.1')\Big\} = 7531'^k$$

$F_W^T = 370^k$ (FROM PG. 5-1)

h_x = HT. OF BOTTOM OF TANK ABOVE BASE OF STRUCTURE
= 1' (IGNORING EDGE TURNDOWN)

F_W' = FORCE COMPONENT OF TANK SUPPORT
$= (0.6) A_a C_W a_x W_{component} = .6(0.4)(1.6)(1) W_{COMP.} = .38\ W_{COMPONENT}$

h_n = HT. OF F_W' ABOVE BASE OF SUPPORT

SECTION	F_W' (K)	h_n (FT)	$F_W' h_n$ ('K)
S1	32.64	.5	16.32
S2	53.69	-1.0	-53.69
S3	12.91	-.66	-8.52
			$\Sigma F_W' h_n = -45.89$

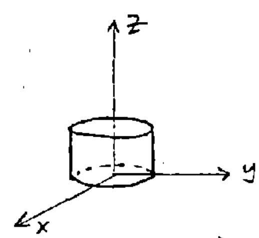
$M_W = 7531'^k + 370^k(1') - 45.89'^k = 7855'^k$

OVERTURNING MOMENT (VERTICAL E.Q.)

$V_W = .6 F_W = .6(.6) A_a C_W a_x (W_T + W_r + W_t + W_{FTG})$
$= .6(.6)(.4)(1.6)(1)(1011^k + 12^k + 46^k + 280.3^k) = 311^k$

$M_z = (13.5')(311^k) = 4196'^k$

MAXIMUM EARTHQUAKE EFFECT:



$M_{E\,MAX} = \sqrt{M_x^2 + M_y^2 + M_z^2}$ (EQ. VII-23)

$M_x = 7855'^k$, $M_y = 0$, $M_z = 4196'^k$

$M_{E\,MAX} = 8905'^k$

NET MOMENT: $M = .75 M_{DL} - M_{E\,MAX}$ (EQ. VII-26)

M_{DL} = RESISTING MOMENT
$= 13.5'(W_t + W_r + W_{FTG} + W_T) = 13.5'(46^k + 12^k + 280^k + 1011^k)$
$= 18216'^k$

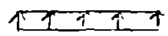
$M = .75(18216'^k) - 8905'^k = 4757'^k$ NO OVERTURNING.

Figure VII D-10 (cont.)

SOIL PRESSURE

a) FROM DEAD LOAD:

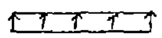
$$1.05\,W_{DL} = 1.05\,(46^k + 12^k + 1011^k + 280^k) = 1416^k$$



$$w_{DL} = 1416^k/\text{AREA} = \frac{1416^k}{\pi(17.5')^2} = 1.472 \text{ KSF}$$

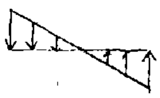
b) FROM SEISMIC LOADING:

(1) AXIAL LOADING



$$w_{\substack{SEIS\\AXIAL}} = \frac{V_W}{\text{AREA}} = \frac{311^k}{\pi(17.5')^2} = .312 \text{ KSF}$$

(2) HORIZONTAL LOADING:



$$f = \frac{M_{E\ MAX}}{S} \qquad S = \frac{\pi r^3}{4} = \frac{\pi(17.5')^2}{4} = 4209 \text{ ft}^3$$

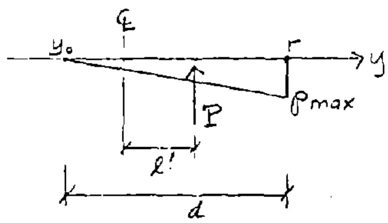
$$f = \frac{8905'^k}{4209\text{ ft}^3} = 2.116 \text{ KSF}$$

$$\text{MAX. SEISMIC PRESSURE} = \sqrt{(2.116)^2 + (.312)^2} = 2.139 \text{ KSF} > w_{DL}$$

THEREFORE, SOME OF AREA IS NOT IN COMPRESSION.

$$\text{EQUIVALENT SEISMIC AXIAL LOAD} = (2.139 \text{ KSF} - 2.116)\,\pi(17.5')^2 = 22.1^k$$

SOIL PRESSURE ON FOUNDATION:



P = TOTAL PRESSURE BETWEEN $y = y_0$ AND r

$P\ell'$ = MOMENT OF P ABOUT ℄ TANK.

$$P = 1416^k + 22^k = 1438^k$$

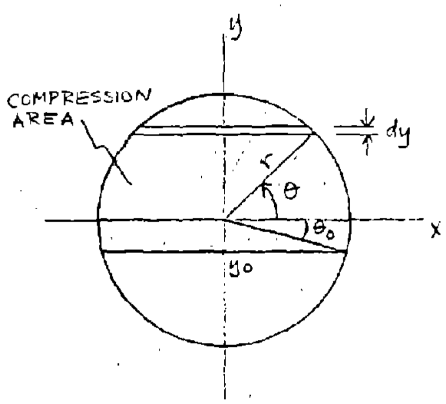
$$P\ell' = M_{E\ MAX}$$

$$\ell' = 8905^k/1438^k = 6.19^k$$

p_{max} = MAX. PRESSURE IN SOIL

$p = p(y)$

WHEN $y = y_0$, $p = 0$; $y = r$, $p = p_{max}$



SOLVE FOR p_{max} AND y_0.

$$P = \int_{y_0}^{r} (2x)\,p\,dy$$

WHERE $dy = r\cos\theta\, d\theta$

$2x = 2r\cos\theta$

p = PRESSURE AT ANY POINT.

Figure VII D-10 (cont.)

$$p = \frac{p_{max}}{r-y_o}(y-y_o) \qquad y = r\sin\theta, \quad y_o = r\sin\theta_o$$

$$= \frac{p_{max}(\sin\theta - \sin\theta_o)}{(1-\sin\theta_o)}$$

$$P = \int_{\theta_o}^{\pi/2}\left[\frac{p_{max}(\sin\theta - \sin\theta_o)}{1-\sin\theta_o}\right]2r\cos\theta \; r\cos\theta \; d\theta$$

$$= \frac{2p_{max}r^2}{1-\sin\theta_o}\int_{\theta_o}^{\pi/2}(\sin\theta - \sin\theta_o)\cos^2\theta \; d\theta$$

$$= \frac{2p_{max}r^2}{1-\sin\theta_o}\left[-\frac{\cos^3\theta}{3}\Big|_{\theta_o}^{\pi/2} - \sin\theta_o\left(\frac{\theta}{2}+\frac{\sin 2\theta}{4}\right)\Big|_{\theta_o}^{\pi/2}\right]$$

$$P = \frac{p_{max}r^2}{6(1-\sin\theta_o)}\left[4\cos^3\theta_o + 6\theta_o\sin\theta_o + 3\sin\theta_o\sin 2\theta_o - 3\pi\sin\theta_o\right] \quad \binom{EQ.}{1}$$

MOMENT OF PRESSURE, $P\ell'$:

$$P\ell' = \int_{\theta_o}^{\pi/2} y(p)(2x)\,dy$$

$$P = \frac{1}{\ell'}\int_{\theta_o}^{\pi/2}(r\sin\theta)\left[\frac{p_{max}(\sin\theta - \sin\theta_o)}{1-\sin\theta_o}\right](2r\cos\theta)\,r\cos\theta \; d\theta$$

$$= \frac{2p_{max}r^3}{\ell'(1-\sin\theta_o)}\int_{\theta_o}^{\pi/2}\sin\theta(\sin\theta - \sin\theta_o)(\cos^2\theta)\,d\theta$$

$$= \frac{2p_{max}r^3}{\ell'(1-\sin\theta_o)}\left[\left(\frac{\theta}{8} - \frac{7\sin 2\theta}{16} + \frac{\sin^3\theta\cos\theta}{4}\right)\Big|_{\theta_o}^{\pi/2} - \sin\theta_o\left(-\frac{\cos^3\theta}{3}\right)\Big|_{\theta_o}^{\pi/2}\right]$$

$$P = \frac{2p_{max}r^3}{\ell'(1-\sin\theta_o)}\left[\frac{\pi}{16} - \frac{\theta_o}{8} + \frac{7\sin 2\theta_o}{16} - \frac{\sin^3\theta_o\cos\theta_o}{4} - \frac{1}{3}\sin\theta_o\cos^3\theta_o\right] \quad \binom{EQ.}{2}$$

SOLVING EQ. 1 AND 2 :

$\theta_o = -.082$ rad ; $y_o = -1.43'$

$p_{max} = 6.48$ KSF

ALLOWABLE BEARING STRESS = 1.33 (5 KSF) = 6.65 KSF

$p_{max} < 6.65$ KSF ✓OK.

SLAB SHALL BE DESIGNED ACCORDING TO ACI-318 USING THE LOADS AND PRESSURES DETERMINED ABOVE.

Figure VII D-10 (cont.)

SUSPENDED PIPE

ASSUME: 16" ϕ DUCTILE IRON PIPE

$F_u = 60$ KSI

$F_y = 42$ KSI

$E = 24 \times 10^3$ KSI

CLASS 53

PIPE FULL OF WATER

70 PSI INTERNAL PRESSURE

PIPE SUPPORTED BY 2^{ND} FLOOR OF 2-STORY BUILDING

FROM MAP, $A_a = 0.40$

TWO-WEEK SHUTDOWN ACCEPTABLE

VELOCITY OF WATER: 6 FT/SEC.

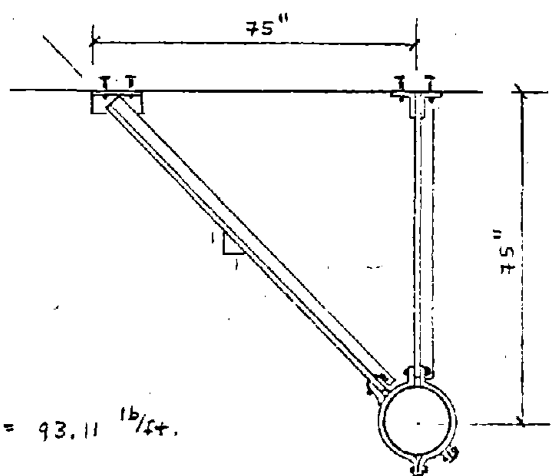
SYSTEM DICTATES MAXIMUM BRACING SPACING OF 25'

PIPE DEFLECTION LIMITED TO $\ell/360$

PIPING SYSTEM SHALL BE RIGID (i.e. FREQUENCY > 20 HZ)

WORKING STRESS DESIGN METHOD

A) TRANSVERSE BRACING



DEAD LOAD

16" ϕ DUCTILE IRON PIPE:

O.D. = 17.4" (RADIUS = 8.70")

CLASS 53: $t = 0.43$"

I.D. = 16.5" (RADIUS = 8.27")

AREA OF PIPE $= \pi[(8.70'')^2 - (8.27'')^2]$

$= 22.92 \ in^2$

ASSUME WT. OF PIPE = 450 $^{lb}/_{ft^3}$

$w_{pipe} = 71.63 \ ^{lb}/_{FT}$

$w_{water} = (62.4 \ ^{lb}/_{ft^3})\pi(8.27 \ in)^2 \frac{1}{144} = 93.11 \ ^{lb}/_{ft}$.

ASSUME WT. OF PIPE CLAMPS & SUPPORTS NEGLIGIBLE

$w_{DL} = .165 \ ^{k}/_{FT}$

EARTHQUAKE LOADS

FOR RIGID SYSTEM,

$F_W = 0.6 \ A_a C_w a_x W$ (EQ. VII-5)

$= 0.6(.40)(1.6)(1.5)(.165 \ ^{k}/_{'})$

$F_W = .095 \ ^{k}/_{'}$

$V_W = .6 F_W$ (SECTION C)

$= .6(.095 \ ^{k}/_{'}) = .057 \ ^{k}/_{'}$

$A_a = 0.40$

$C_w = 1.6$

$a_x = 1 + \frac{1}{2} = 1.5$

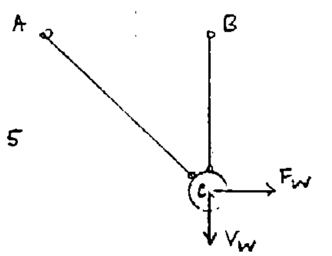


Figure VII D-11 Suspended pipe design example

COMBINING LOAD EFFECTS,

MAX. VERTICAL LOAD $= 1.05 Q_D + Q_{E\,MAX}$ (EQ. VII-24)

$$= 1.05(.165\ ^{K}/_{'}) + \sqrt{0 + (.057\ ^{K}/_{'})^2 + 0}$$

$$= .230\ ^{K}/_{'}$$

MAX. HORIZ. LOAD $= 1.05 Q_D + Q_{E\,MAX}$

$$= 0 + \sqrt{0 + 0 + (.095\ ^{K}/_{'})^2}$$

$$= .095\ ^{K}/_{'}$$

SPACING OF SUPPORTS

3 CRITERIA TO BE CONSIDERED: 1) DEFLECTION : $\Delta \le \ell/360$

2) BENDING : $f \le F_y/1.5$

(from ANSI A21.50-1976)

3) RIGIDITY : $T \le .05$ SEC

1) DEFLECTION

ASSUME PIPE IS HINGED AT SUPPORTS - CONSERVATIVE

$$\frac{\ell}{360} = \frac{5 w \ell^4}{384 EI} \qquad I = \frac{\pi}{4}\left[(8.70")^4 - (8.27")^4\right] = 825.8\ in^4$$

$$\ell^3 = \frac{384\ EI}{5w\ (360)} = \frac{.213\ (24 \times 10^3\ ksi)(825.8\ in^4)}{(.230\ ^{K}/_{'})(1/12)}$$

$$\ell = 604"$$

$$\ell \le 50.3'$$

2) BENDING

$$f = \frac{M}{S} \qquad M = \frac{w\ell^2}{8} = \frac{(.230\ ^{K}/_{'})\ \ell^2}{8} = (.029\ ^{K}/_{'})\ \ell^2$$

$$S = \frac{825.8\ in^4}{8.70\ in} = 94.91\ in^3$$

$$f = \frac{(.029\ ^{K}/_{'})\ell^2}{94.91\ in^3}\left(\frac{1}{12}\right) = (2.55 \times 10^{-5}\ ^{K}/_{in^4})\ \ell^2$$

$$f_a = 1.33\ (.6 F_y) = .798\ (42\ ksi) = 33.5\ ksi$$

$$33.5\ ksi = (2.55 \times 10^{-5}\ ^{K}/_{in^4})\ \ell^2$$

$$\ell = 1146"$$

$$\ell \le 95.5'$$

3) RIGIDITY

$$\omega = \frac{\pi^2}{\ell^2}\sqrt{\frac{EIg}{w}}$$

FROM STRUCTURAL ENGINEERING HANDBOOK FOR CIVIL ENGINEERS, MERRITT, F.S., ED.; 1968, p. 6-112 (206)

$$T = 2\pi/\omega = 2\ell^2/\pi\sqrt{EIg/w}$$

where ω = ANGULAR FREQUENCY

$$\ell^2 = .5\pi T\sqrt{\frac{EIg}{w}}$$

$w = w_{DL}$

Figure VII D-11 (cont.)

$$\ell^2 = .5\pi\,(.05\ \text{sec})\sqrt{\frac{(24\times10^3\ \text{ksi})(825.8\ \text{in}^4)(32.2\ \text{FT/s}^2)}{(.165\ \text{K/'})}}\left(\frac{1}{12}\right)$$

$\ell \le 20.2'$ THIS CRITERION GOVERNS. <u>USE 20' SPACING</u>

<u>TENSILE FORCES ON BRACES:</u>

$$T_{BC} = 1.05\,Q_D + Q_{E\,MAX} \quad (\text{EQ. VII-24})$$

$$= \left[1.05\,(.165\ \text{K/'}) + \sqrt{0 + (.057\ \text{K/'})^2 + (.095\ \text{K/'})^2}\right](20')$$

$$= 5.68^K$$

$$T_{AC} = Q_{E\,MAX} = (20')\sqrt{(.095\cos 45)^2 + 0 + 0} = 1.34^K$$

<u>SUMMARY:</u> FOR 20' BRACING SPACING,

DESIGN BRACE AC FOR 1.34^K TENSION

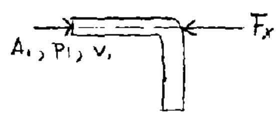
1.34^K COMPRESSION

DESIGN BRACE BC FOR 5.68^K TENSION

B) <u>LONGITUDINAL BRACING ADJACENT TO EXPANSION JOINT</u>

<u>EFFECT OF MOVING WATER</u>

HORIZONTAL FORCES ON WATER:



APPLY IMPULSE-MOMENTUM THEOREM:

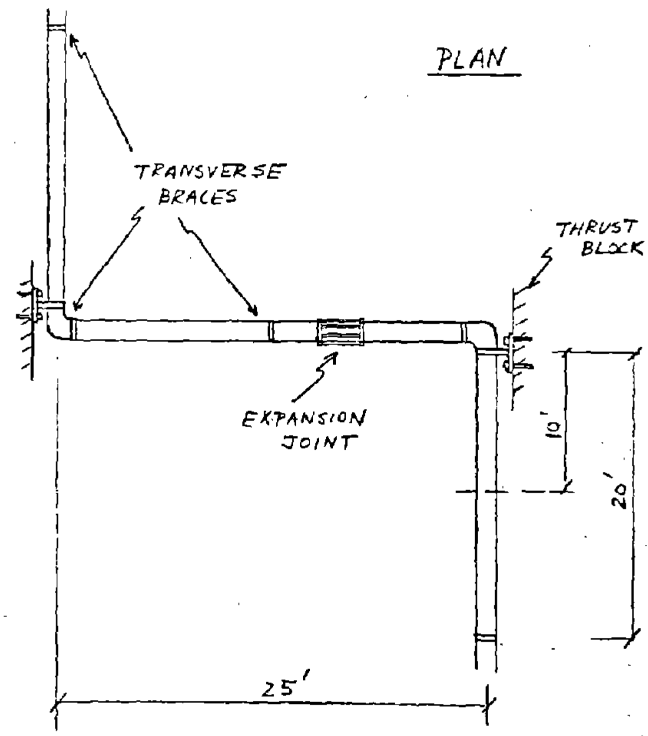
$$P_1A_1 - F_x = \rho Q(-v_1)$$

$$F_x = P_1A_1 + \rho Q v_1$$

$$P_1 = 70\ \text{psi}$$

$$A_1 = \pi r^2 = \pi(8.25'')^2 = 213.8\ \text{in}^2$$

$$v_1 = 6\ \text{ft/s}$$



$$F_x = (70\ \text{psi})(213.8\ \text{in}^2) + (62.4\ \text{lb/ft}^3)(8.91\ \text{ft}^3/\text{s})(6\ \text{ft/s}) = 18.30^K$$

HORIZ. FORCE ON THRUST BLOCK $= 18.30^K$

Figure VII D-11 (cont.)

EARTHQUAKE EFFECT

$w_{DL} = .165\ ^{K}/_{'}$

LENGTH OF PIPE $= 12.5' + (10') = 22.5'$

$W = 22.5'(.165\ ^{K}/_{'}) = 3.71^{K}$

$F_W = 0.6\ A_a\ C_w\ a_x\ W$ (EQ. VII-5)

$= 0.6\,(.40)(1.6)(1.5)(3.71^{K})$

$F_W = 2.14^{K}$

COMBINING LOAD EFFECTS:

R = REACTION AT CRITICAL THRUST BLOCK

$R = 1.05\,Q_D + Q_{E\,MAX}$ (EQ. VII-24)

$= 1.05\,(18.30^{K}) + 2.14^{K} = 21.36^{K}$

DESIGN THRUST BLOCK AND CONNECTION FOR FORCE OF 21.36^{K}

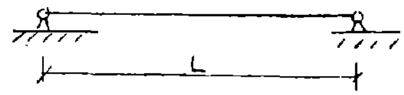
ALLOWABLE STRESSES MAY BE INCREASED BY 1/3 (AISC 1.5.6 (204))

Figure VII D-11 (cont.)

MAXIMUM UNBRACED LENGTHS FOR RIGID PIPE

L = DISTANCE BETWEEN VERTICAL SUPPORTS

ASSUME PINNED END CONDITION;
PIPE FULL OF WATER.



STEEL PIPE: ASTM A53; TYPE S
GRADE B

DUCTILE IRON PIPE: THICKNESS CLASS 53

SPAN LENGTH IS GOVERNED BY RIGIDITY CRITERION:

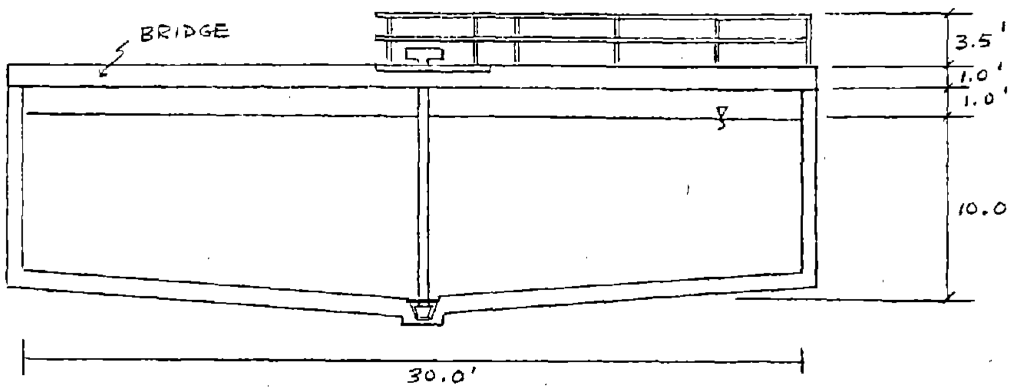
$$L = \left(0.50\,\pi\,T\sqrt{\frac{EIg}{w}}\right)^{1/2}$$

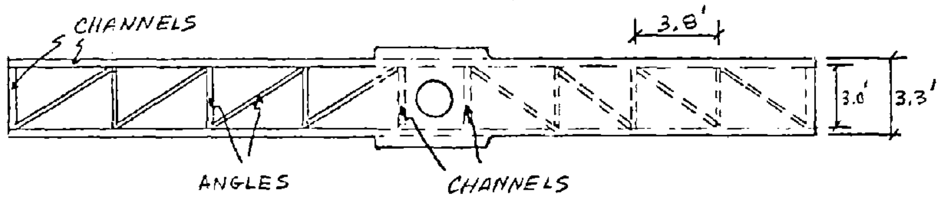
WHERE T = PERIOD = .05 SEC
E = MODULUS OF ELASTICITY OF PIPE
I = MOMENT OF INERTIA OF PIPE
w = DEAD LOAD WEIGHT (PIPE AND WATER) PER UNIT LENGTH

	MAXIMUM SPAN, L		
DIAMETER (INCHES)	STEEL PIPE, A53, GR.B STD. WT.	STEEL PIPE, A53, GR B EXTRA STRONG	DUCTILE IRON PIPE, CLASS 53
3	10'-3"	10'-5"	10'-8"
4	11'-6"	11'-9"	11'-8"
6	13'-9"	14'-2"	13'-9"
8	15-6"	16'-1"	15'-6"
10	17'-1"	17'-7"	16'-9"
12	18'-4"	18'-11"	18'-0"
14	19'-0"	19'-7"	19'-3"
16	20'-0"	20'-8"	20'-2"
18	20'-9"	21'-7"	21'-1"
20	21'-7"	22'-6"	21'-10"
24	22'-11"	24'-0"	
26	23'-7"	24'-8"	

Figure VII D-11 (cont.)

CLARIFIER BRIDGE





ASSUME :

STRUCTURAL STEEL BRIDGE SPANNING 30.0' DIAMETER REINFORCED CONCRETE CLARIFIER, WITH DIMENSIONS AS SHOWN.

FROM MAP, $A_a = 0.15$

2 WEEK SHUTDOWN ACCEPTABLE

RAILING AND CHECKERPLATE EXTENDING OVER HALF THE BRIDGE.

BRIDGE NOT SUPPORTED AT CENTER BY SHAFT.

USE WORKING STRESS METHOD.

LOADING

1) DEAD LOADS

a) CHANNELS: ASSUME C12 × 20.7 WT = .021 K/FT. OF CHANNEL.

b) ANGLES: ASSUME L2½ × 2½ × ¼ , WT = .004 K/FT. OF ANGLE
DISTRIBUTING TOTAL WEIGHT OF ANGLES AND 4 SHORT CHANNELS UNIFORMLY ALONG BRIDGE: $w_{DL} = .017$ K/'

c) EQUIPMENT MOUNTED AT CENTER: $P_{DL} = 1.5^{K}$ CONCENTRATED LOAD

d) CHECKERPLATE : ASSUME 3/16" PL

$$w_{DL} = (3/16")(.490\ K/')(\tfrac{1}{12})(3.3') = .025\ K/'$$

Figure VII D-12 Clarifier bridge design example

e) RAILING: ASSUME $1\frac{1}{2}"\phi$ STRUCTURAL STEEL PIPE, $WT = .003\ ^{K}/_{\prime}$

LINEAR FT OF RAILING $= 4(16') + 10(3.3') + 2(4') = 107'$

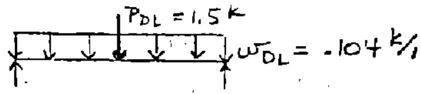
TOTAL WT $= (107')(.003\ ^{K}/_{\prime}) = .321^{K}$

$w_{DL} = .321^{K}/16' = .020\ ^{K}/_{\prime}$

FOR DESIGN, ASSUME THAT CHECKERPLATE AND RAILING EXTEND OVER ENTIRE BRIDGE TO SIMPLIFY LOADING — CONSERVATIVE

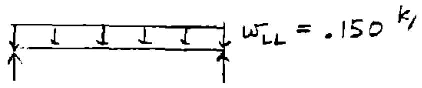
UNIFORM DL $= w_{DL} = 2(.021\ ^{K}/_{\prime}) + .017\ ^{K}/_{\prime} + .025\ ^{K}/_{\prime} + .020\ ^{K}/_{\prime}$

$= .104\ ^{K}/_{\prime}$



2) LIVE LOAD – ASSUME 50 PSF

$w_{LL} = (.05\ ^{K}/_{FT^2})(3.0') = .150\ ^{K}/_{\prime}$



3) SEISMIC LOADS

SUBMERGED SHAFT HAS A BEARING ON BRIDGE AT CENTER; THUS LATERAL SEISMIC LOADS ON THE SHAFT WILL IMPART A FORCE ON THE BRIDGE.

TOTAL TRIBUTARY MASS = MASS OF STRUCTURE $+ m_a H$

m_a = ADDED MASS OF LIQUID ACTING WITH SHAFT (PER FOOT)

$=$ (ADDED MASS RATIO) $W/g\ \pi r_o^2$ (EQ. VII-19)

H = HEIGHT OF SHAFT THAT IS SUBMERGED.

WT OF STRUCTURE; ASSUME $6"\phi$ STD. STRUCTURAL STEEL PIPE

$WT = 18.97\ ^{\#}/_{\prime}$, O.D $= 6.625"$, O.R. $= 3.313"$

TRIB. WT. OF STRUCTURE $= (18.97\ ^{\#}/_{\prime})(12') = .228^{K}$

$\frac{r_o}{H} = \frac{3.313"}{10'}\left(\frac{1}{12}\right) = .03$

FIND ADDED MASS RATIO AT 3 LEVELS IN THE TANK:

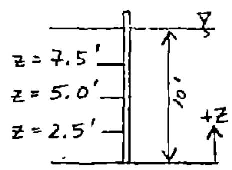
AT $z = 2.5'$: $z/H = .25$

ADDED MASS RATIO $\simeq 1.0$ (FIG. VII-9)

$m_a = (1.0)\frac{W}{g}\pi r_o^2$

$= (1.0)(62.4\ ^{lb}/_{ft^3})\pi(3.313")^2\frac{1}{144}$

$= 14.94\ ^{\#}/_{\prime}$



AT $z = 5.0'$: $z/H = .25$, ADDED MASS RATIO $\simeq 1.0$

$m_a = 14.94\ ^{\#}/_{\prime}$ (SAME AS ABOVE)

AT $z = 7.5'$: $z/H = .75$, ADDED MASS RATIO $\simeq .96$

$m_a = (.96)(62.4)\pi(3.313)^2\frac{1}{144} = 14.34\ ^{\#}/_{\prime}$

Figure VII D-12 (cont.)

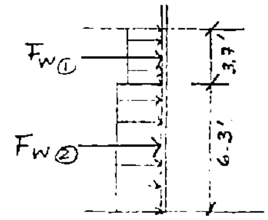
$(m_a H)_{①} = (14.34\ ^{\#}/{}') (3.7') = 53.07^{\#}$

$F_{W①} = 0.6 A_a C_W a_x W \quad (\text{EQ. VII-5})$

$= 0.6(.15)(1.6)(1)(m_a H)_{①} = .008^{K}$

$(m_o H)_{②} = (14.94\ ^{\#}/{}')(6.3') = 94.12^{\#}$

$F_{W②} = 0.6(.15)(1.6)(1)(m_o H)_{②} = .014^{K}$

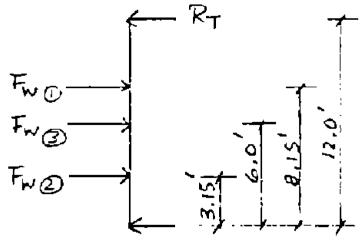


$F_{W③}$ = E.Q. FORCE ON SHAFT DUE TO ITS OWN MASS

$= 0.6 A_a C_W a_x W$

$= 0.6(.15)(1.6)(1)(.228^{K}) = .033^{K}$

REACTION ON BRIDGE FROM $F_{W①}$, $F_{W②}$, $F_{W③}$:

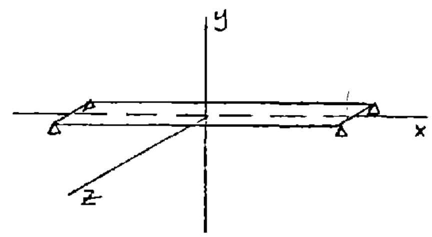


$R_T = \frac{1}{12'}\left[3.15'(.014^{K}) + (6')(.033^{K}) + (8.15')(.008^{K})\right]$

$R_T = .026^{K}$

VERTICAL SEISMIC FORCES ON THE SHAFT AND WATER ARE NOT TRANSMITTED TO THE BRIDGE.

(1) SHEAR AT SUPPORTS



$Q_x = \frac{1}{4}(R_T + F_{W_B})$

Q_x = SHEAR ON EACH OF FOUR BRIDGE BEARINGS DUE TO X-Y EQ.

F_{W_B} = SEISMICALLY INDUCED FORCE DUE TO MASS OF BRIDGE

$= 0.6 A_a C_W a_x W = 0.6(.15)(1.6)(1)\left[(.104\ ^{K}/{}')(30') + 1.5^{K}\right]$

$= .665^{K}$

$Q_x = \frac{1}{4}(.026^{K} + .665^{K}) = .173^{K}$

$Q_z = Q_x \quad , \quad Q_y = 0$

$Q_{E\ MAX} = \sqrt{Q_x^2 + Q_y^2} = \sqrt{2}\,(.173^{K}) = .245^{K}$

$Q_D = 0$

DESIGN EACH CONNECTION FOR $.245^{K}$ SHEAR

Figure VII D-12 (cont.)

(2) UPLIFT AT SUPPORTS

Q = UPLIFT ON EACH BRIDGE BEARING DUE TO E.Q.

$Q_x = Q_z = 0$

VERTICAL SEISMIC FORCE: $V = .6\,F_{W_B} = .6(.665^k) = .399^k$

$Q_{E\ MAX} = \frac{1}{4}V = .100^k$

$Q_D = \frac{1}{2}\left[\frac{1.5^k}{2} + \frac{(.104\ ^k/_')(30')}{2}\right] = 1.16^k$

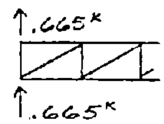
$Q = .75\,Q_D - Q_{E\ MAX}$ (EQ. VII-26)

$= .75(1.16^k) - .100^k$

$= .770^k$ NO UPLIFT ON BRIDGE BEARINGS.

(3) AXIAL FORCES IN TRANSVERSE & DIAGONAL TRUSS MEMBERS

a) IN TRANSVERSE ANGLE:



$Q_x = Q_y = 0$

$Q_z = Q_{E\ MAX} = 1.33^k$

ALLOWABLE TENSILE FORCE

$= A_n F_t$

$= (1.01\ in^2)(22\ ksi)$

$= 22.25^k > 1.33^k$ ✓OK.

A_n = NET AREA

$F_t = 22$ ksi

ASSUME $A_n = .85\,A_g = .85(1.19\ in^2)$

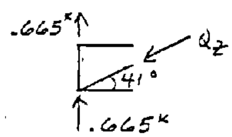
$= 1.01\ in^2$

ALLOWABLE COMPRESSIVE FORCE:

$k\ell/r = \frac{(1)(3.3')}{(.491'')}(12) = 80.7 \Rightarrow F_a = 15.24$ ksi

ALLOW. FORCE $= (15.24\ ksi)(1.19\ in^2) = 18.14^k > 1.33^k$ ✓OK

b) IN DIAGONAL ANGLE:



$Q_x = Q_y = 0$

$Q_{E\ MAX} = Q_z = \frac{2(.665^k)}{\sin 41°}$

$= 2.03^k < 22.3^k$ TENSION ✓OK

$< 18.1^k$ COMPRESSION ✓OK.

(4) CHANNELS

a) WEAK AXIS BENDING (z-z E.Q.): NEGLECT

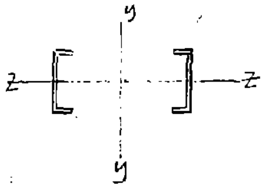
b) STRONG AXIS BENDING (y-y E.Q., DL, AND LL):

$M_{E\ max} = M_y = \frac{V\ell}{4} = \frac{(.339^k)(30')}{4} = 2.54^{'k}$ $(M_x = M_z = 0)$

$M_{LL\ max} = \frac{w_{LL}\ell^2}{8} = (.150^k)(30')^2/8 = 16.88^{'k}$

$M_{DL\ max} = P_{DL}\,\ell/4 + \frac{w_{DL}\ell^2}{8} = \frac{(1.5^k)(30')}{4} + \frac{(.104^k)(30')^2}{8} = 34.7^{'k}$

Figure VII D-12 (cont.)



$$I_{zz} = 2(129\ in^4) = 258\ in^4 \quad (2\ \text{channels})$$

$$S = 2(21.5\ in^3) = 43\ in^3$$

$$f_{(1.05D+L)} = \frac{M}{S} = \frac{1.05(34.7'^k) + 16.88'^k}{43\ in^3}(12) = 14.88\ ksi$$

$$f_{SEISMIC} = \frac{2.54'^k}{43\ in^3}(12) = .71\ ksi$$

ALLOWABLE BENDING STRESS:

$$F_b = \frac{(12 \times 10^3)C_b}{\ell\ d/A_f} \qquad \text{(AISC 1.5-7 (540)) FOR 1 CHANNEL}$$

ℓ = UNBRACED LENGTH = 3.8'

BUT NOT MORE THAN $.6F_y = 22$ ksi

$$F_b = \frac{2(12 \times 10^3)(1)}{(3.8')(8.13)}\left(\frac{1}{12}\right) = 32.4 > 22\ ksi \quad \therefore\ F_b = 22\ ksi$$

c) <u>AXIAL FORCES</u> (Z-Z EQ.):

$$M_{MAX} = \frac{(R_T + F_{W_B})\ell}{4} = \frac{(.026^k + .665^k)(30')}{4} = 5.18\ '^k$$

$$\varphi_z = \frac{5.18'^k}{3.3'\ (AREA)} \qquad AREA = 6.09\ in^2$$

$$= .26\ ksi$$

ALLOWABLE COMPRESSIVE STRESS:

$$\frac{K\ell}{r} = \frac{(1)(3.8')}{.799''}(12) = 57.1 \Rightarrow F_a = 17.62\ ksi$$

d) <u>AXIAL FORCES</u> (X-X EQ.):

$$\varphi_x = \frac{.665^k}{6.09\ in^2} = .11\ ksi \quad , \quad F_a = 17.62\ ksi.$$

e) CHECK <u>COMBINED AXIAL COMPRESSION & BENDING</u> AS PER AISC 1.6.1 (204)

USING RATIOS : $\frac{\text{STRESSES}}{\text{ALLOW. STRESSES}}$

$$\varphi_{E\ MAX} = \sqrt{\underbrace{\left(\frac{0.71}{22}\right)^2}_{\substack{\text{BENDING}\\ \text{y-y EQ}}} + \underbrace{\left(\frac{0.26}{17.62}\right)^2}_{\substack{\text{COMPR.}\\ \text{z-z E.Q.}}} + \underbrace{\left(\frac{0.11}{17.62}\right)^2}_{\substack{\text{COMPR.}\\ \text{x-x E.Q.}}}} = .044$$

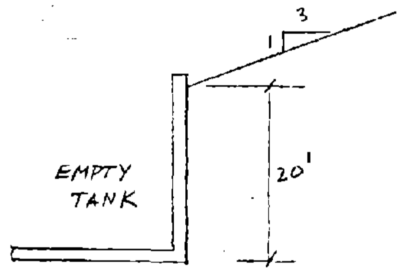
$$\underbrace{\frac{14.88}{22}}_{\substack{\text{COMPR.}\\ \text{1.05 DL + LL}}} + \varphi_{E\ MAX} = .720 < \underbrace{1.33}_{\substack{\text{INCREASED BY 1/3}\\ \text{AS PER AISC 1.5.6 (204)}}} \quad \checkmark OK$$

Figure VII D-12 (cont.)

RETAINING WALL

ASSUME:

REINFORCED CONCRETE RETAINING WALL
SOFT CLAY AND SAND ($S = 1.5$)
FROM MAP, $A_a = 0.20$
2 WEEK SHUTDOWN ACCEPTABLE
WT. OF SOIL = $\lambda = 110\ \#/ft^3$
$\phi = 35°$
USE STRENGTH METHOD



$$A_a S C_s = (.20)(1.5)(2) = .6 > 0.175$$

∴ INCREASE IN ACTIVE EARTH PRESSURE, F_s, FROM SEISMIC ACCELERATIONS MUST BE ACCOUNTED FOR (PG. VII-15).

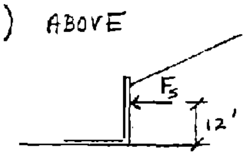
$$F_s = \frac{\lambda H^2 A_a S C_s}{6 \tan \phi}\left(1 + \frac{6i}{100}\right) \qquad (\text{EQ. VII-20})$$

$$i = \tan^{-1} \frac{1}{3} = 18.43°$$

$$= \frac{(110\ ^{lb}/_{ft^3})(20')^2(.20)(1.5)2}{6 \tan 35°}\left[1 + \frac{6(18.43)}{100}\right]$$

$F_s = 13,232\ ^{lb}/_{FT}$ OF WIDTH

$F_s = 13.232\ ^{k}/_{FT}$ APPLIED AT POINT $0.6H$ (12') ABOVE BASE OF WALL.



THE LOAD EFFECTS FROM F_s WOULD BE COMBINED WITH THE LOAD EFFECTS OF THE VERTICAL ACCELERATION V_s AS DETERMINED BY USING SECTION C, WHEREIN THE EFFECTIVE WEIGHT WOULD BE THE WEIGHT OF THE WALL AND ANY EARTH OVER THE WALL FOOTING (IF ANY). THE MAXIMUM EARTHQUAKE EFFECTS WOULD BE DETERMINED USING EQUATION VII-23 AND WOULD BE COMBINED WITH THE DEAD LOAD EFFECTS USING VII-24 AND VII-25.

Figure VII D-13 Retaining wall design example

CHAPTER VIII

EMERGENCY RESPONSE AND PLANNING

Unlike most natural disasters, an earthquake typically occurs with no advance warning. Although total preparation for its arrival is impossible, the potentially devastating impacts can be somewhat minimized through careful planning and preparation for rapid and efficient emergency response.

Planning for an earthquake can involve defining the critical elements of water and wastewater facilities, physically modifying the structures, organizing a chain of command and communications network, and thoroughly familiarizing personnel with the "action features" of the emergency plan including proper response measures. The response itself includes providing emergency relief, assessing damages, restoring disrupted services, and repairing damaged equipment, all of which may require outside assistance.

Specific methods for preparing an emergency plan will be described in the second half of this chapter. Before an effective plan can be devised, a prior understanding of the goals and nature of the response is crucial. The basic structure of an emergency response will be discussed below, followed by examples of specific response procedures.

Although the following section will only consider emergency response and planning for earthquakes, an overall emergency plan for a water or sewer utility might encompass other manmade or natural disasters, i.e., nuclear attacks, civil disorders, floods, hurricanes, etc. Incorporation of several different possible disaster events and their effects into an overall emergency operations plan is more efficient than having several individual plans.

A. EMERGENCY RESPONSE

The primary goals of an earthquake response program are to prevent threat to or loss of life and property, to provide essential water and wastewater services by maintaining the operation of present facilities or providing alternative systems, and to locate and repair damages (207). Although it would be convenient to organize the associated plan according to sequential activities such as damage location, decision-making and restoration, the realistic emergency situations that arise after an earthquake dictate that the response be structured, instead, on a priority basis.

The emergency period can thus be considered to consist of three phases: the immediate emergency phase, the sustained emergency phase, and the repair phase (208). These phases are overlapping, and their duration depends on the severity of damage sustained. In general, the immediate emergency phase will be concluded within hours of the earthquake, the sustained emergency phase within days and the major part of the repair phase within weeks or even months.

The response procedures presented in this chapter were developed from descriptions of how utilities and local governments responded to actual earthquake occurrences, and from response procedures delineated in emergency plans of selected localities.

The immediate emergency phase, which begins with the occurrence of the earthquake, consists of two basic activities: making a preliminary survey of damages to critical facilities and taking emergency measures (209). Critical elements of water and wastewater facilities should be defined in the emergency plan. In general, highest priority should be given to the inspection of dams and the immediate implementation of emergency measures to prevent dam failure. Assuring the immediate availability of water for firefighting is also of utmost priority. Although reporting damages to a central office is important throughout most of the response period, operational changes required to prevent serious danger to life, extensive property damage, and significant damage to critical portions of the system may have to be made prior to reporting, provided that these changes are reported as soon as possible (207). Procedures for responding to the immediate dangers that may arise should be described in the emergency plan to minimize the amount of independent decision-making that must be made under stressful conditions.

If local manpower and resources cannot properly respond to the immediate emergency, voluntary organizations should be called upon and mutual aid agreements should be executed immediately. Note, however, that unless special training has been given to these outside groups, they will be unfamiliar with the system and the required response operations.

Once the immediate remedial actions necessary to protect life, property and critical facilities have been performed, priority should be given to restoring water and wastewater services to as close to normal operations as possible (sustained emergency phase). This phase entails a continued and more detailed inspection of water and wastewater facilities. On the basis of the incoming information, a detailed assessment of the nature, severity, location and costs of damages can be made (210).

Of greatest priority during this phase is a determination of the quantity of potable water available relative to water needs (211). If normal water supplies are insufficient and if the system is damaged beyond immediate repair, provisions should be made for alternative systems, either on a temporary basis (e.g., tank trucks) or on a semi-permanent basis (e.g., surface distribution systems). Provisions should be made for assuring the safety of drinking water from partially damaged and temporary systems. If necessary, temporary sanitary facilities should also be provided during this phase.

The repair phase begins with the known condition of the water and wastewater facilities. Because essential services will have been restored prior to this phase, at least on a temporary basis, the overall repair process may not be correctly labelled an emergency procedure. Nevertheless, because the repair process consumes much of the post-earthquake response effort, it will be discussed as part of the emergency response.

On the basis of the damage assessment, priorities can be established among needed repairs, and a repair schedule devised. Planning the repair effort will entail reviewing the extent of damage, determining the level of effort necessary for its repair, and deciding whether the repair will be achieved by utility forces or by outside contractors.

It should be noted that if the damage is widespread, such as that sustained to an extensive water distribution or sewer system, locating the damage may extend into this phase. In this case, detailed damage inspection and repair may continue for several months.

IMMEDIATE EMERGENCY PHASE

Immediately following an earthquake, efforts must be directed at preventing loss of life and serious property damage. Because the major continuing threats to life and property are imposed by fires and floods (212), primary targets for immediate emergency response are water system facilities needed to supply water for firefighting, and dams. Other system elements which, if damaged, could present a hazard to treatment plant personnel should also receive immediate attention. These high-priority facilities should be identified in the emergency plan.

The ideal response sequence immediately following an earthquake would be:

1. Inspect critical facilities and report their condition to a control center
2. Make emergency repairs or provide alternate emergency operations
3. Communicate with other agencies to obtain their assistance or to keep them informed

Unfortunately, it is not always possible to maintain this degree of order under extreme emergency conditions. It is unduly optimistic to expect a control center to be fully activated during the first few hours after an earthquake. Combined with potential disruptions in communication systems, control centers may not be able to organize and direct all immediate activities (212). Inspection personnel must be prepared to personally initiate any operational changes needed to prevent loss of life or extensive property damage. This requires a thorough evaluation of potential problems prior to the earthquake along with the development of guidance for field personnel.

Specific immediate needs for inspection and emergency measures cannot be realistically prescribed in this report for individual utilities. However, target areas can be identified on the basis of analysis of each system, past experience and common sense. High-priority areas and examples of immediate emergency response procedures are discussed below.

Immediately following an earthquake, the Metropolitan Water District of Southern California (MWDSC) directs on-duty personnel at dams, water treatment plants, and pumping stations to check their respective installations for damage which could disrupt operations (207). Of primary importance is a complete visual inspection of dams including the dam embankment, transverse drain weir structures, outlet conduit drain sump pump structures, and piezometer levels (207, 211). In the event of extensive damage to a dam, immediate consideration is to be given to emergency draindown of the impoundment or, if necessary, evacuation of residents below the dam. Although the evacuation itself is not normally the responsibility of the water facility personnel, their recommendations to the appropriate civil authorities should be very rapid. This requires effective, prearranged channels of communication with the local and/or regional authorities responsible for public safety (e.g., police, fire department, civil defense, etc.).

An example of prompt and efficient emergency response was provided by the Los Angeles Department of Water and Power (LADWP) following the 1971 San Fernando earthquake. The upstream face of the Lower Van Norman Reservoir dam had suffered a major slide, and the top of the remaining embankment was about 6' above the water surface. The east outlet tower, which had sheared 20' above its base, had disappeared under water. Within nine minutes of the earthquake, which occurred on February 9 at 6:01 a.m., the reservoir keeper had viewed the dam and determined that there was a problem. Inspection, safety and design engineers were contacted immediately, and the dam was inspected for damage. By 6:30 a.m., steps were taken to increase the normal outflow from the reservoir by spilling water into flood control channels, the Los Angeles River and the Tujunga Spreading Grounds, and by transferring water to other storage reservoirs. By 8:55 a.m., the Chief Engineer of the LADWP had recommended to the Chief of the Police Department and the City's Disaster Committee that the population in the area below the Van Norman Reservoir complex be evacuated because of extensive damage to the dams. In the meantime, activities were being coordinated among high-level engineers in the office of the Chief Engineer. Field reports were monitored on reservoir levels, outflows, turbidities and all emergency repair activities; Executive Command activities were continued there on a round-the-clock basis until February 16 (016).

The MWDSC instructs personnel on duty at pumping plants to inspect buildings, pumping equipment, auxiliary structures, electrical switch-racks, delivery lines, transformers, and transmission lines (207,211). Because pumping facilities must operate if water is to be delivered for fire fighting, emergency power must be immediately provided if the facility is found to be otherwise undamaged.

The MWDSC (207) directs personnel on duty at water treatment plants to immediately evaluate the plants' condition in terms of power status, influent and effluent flows and chlorination system integrity. Because damage to a chlorination system can be immediately threatening to treatment plant personnel, a careful assessment of chlorine leaks or other disturbance is of primary importance (213). In Managua, for example, the 1972 earthquake caused chlorine cylinders to fall, breaking connections and producing chlorine leaks which filled the chlorination room with gas. Alternating technical personnel equipped with protective masks shut off the cylinder valves until all leaks were stopped. The same day, all broken connections were restored; the entire chlorinating operation was fully restored on the next day (034).

If damage to the water treatment facility is extensive and the facility is so equipped, consideration should be given to bypassing untreated water through the plant for firefighting, although chlorination is recommended to limit contamination of the distribution lines and control the impact of possible human consumption (039). A major action such as this, however, which affects not only the quantity but the quality of the water delivered, should not be taken at the discretion of the treatment plant operator; county or state health department officials must be contacted first. In the event of a treatment plant bypass, the public must immediately be instructed to boil water before consumption.

One of the greatest dangers following an earthquake is fire, and the ability to fight fires will largely depend on the integrity of the water distribution system. Breaks in the distribution system can first be located by telemetered pressure data, if available. The distribution system should be inspected by personnel who have been assigned patrol routes. Initially, leaks should be sought in areas where ground disturbance is observed such as areas suffering from slides, differential settlement, and ground cracks. Leaks may be spotted as boils on the ground surface and flows of water in areas that are normally dry. Areas most likely to experience fire damage are small localized zones that are supplied by only a few water mains, where one or two pipeline failures or a power outage at a pumping plant could completely isolate the area. Larger zones, in which water could be rerouted through parallel mains and multiple storage facilities, are less vulnerable (129).

Because maintaining an adequate water supply for firefighting is crucial, primary consideration should be given to stopping uncontrolled water flowing from broken mains by isolating damaged sections with valves. Of course, this will make water unavailable in some localized zones; the fire department should be notified immediately. Isolating damaged sections with valves will also allow pressure to be maintained in undamaged sections. Damage to the distribution section might necessitate rerouting of water, reducing pressure, or informing consumers to curtail water consumption (120).

Following the 1971 San Fernando earthquake, some higher elevation distribution systems were without water because of trunk line breaks. To increase the availability of water for firefighting, the Los Angeles Fire Department provided 20 pumpers. These were sited where the "wet" system (standard water supply system) was close to a "dry" system (maintained without water until flow was required) and where hydrants in both systems were near each other. The pumpers transferred water from the wet to the dry system through fire hose; this arrangement was used for up to 10 days after the earthquake (214).

Three other methods of providing emergency water for firefighting include water trucks, rapid temporary bypasses such as quick-fit irrigation piping, and pumping from nearby stored-water sources such as swimming pools, ponds, lakes, industrial water storage systems, or the ocean (215).

If damage is so excessive that local areas cannot properly respond in terms of inspection and/or immediate repair, a quick assessment should be made of available assistance from other public work agencies in relation to personnel expertise and equipment. Mutual aid agreements should be implemented immediately. These types of decisions will be made by the control center on the basis of information received by inspection personnel. To enable such decisions to be made rapidly, the State of California recommends that descriptions of the occurrence, nature and extent of damage be reported to the control center within four hours (210). Note that arrangements for aid from other agencies or utilities should be made as part of the planning process.

SUSTAINED EMERGENCY PHASE

Sustained emergency operations, which begin when threats from floods and fires have subsided, are aimed at restoring life support services to the affected area. Activities will include supplying potable water and preventing prolonged unsanitary conditions potentially leading to transmission of disease. These goals can be achieved through the rapid repair of water and wastewater facilities or, in the event of extensive damage, the provision of alternate systems.

Selecting suitable restoration techniques calls for prompt and continuing damage reports, assessments of present system capacities and users' demands for services, assessment of available resources and estimated time for repairs, and a prediction of the duration of the emergency state. If severe damage has been sustained, damage inspection and assessment can extend well into the succeeding "repair" phase. Although these activities are discussed in the "sustained emergency" and "repair" phases as they relate to the respective goals of each, it should be noted that they are performed continuously during the earthquake response period.

In contrast with activities conducted during the immediate emergency phase, which are often based on the judgement of individual field personnel, the overall direction of the restoration operation should be provided by the fully-operating control center.

Damage Surveys

Inspection of damages to critical facilities will have been initiated immediately following the earthquake occurrence. Damage surveys must continue during the sustained emergency phase, and enough information must be procured to enable the control center to determine the systems' available functional capacities and the estimated repair effort in terms of equipment, personnel, and time.

A pre-earthquake vulnerability assessment and functional analysis of water and wastewater systems will indicate priority damage survey efforts. Of great importance will be an inspection of the distribution system to determine its integrity, its ability to retain the water supply and to maintain pressure, and the level of pressure (126). The extent of damage to power facilities and other essential supplemental facilities must also be determined.

In general, damage to individual components of water and wastewater treatment plants can be located more easily than damage to water distribution and sewer lines. Some procedures for inspecting lifelines which have met with considerable success during past earthquakes are described below.

Experience in California, Alaska and elsewhere has shown that an effective sewer inspection program consists of visual inspection, rodding, and televising or photographing. Visual inspection is performed soon after the earthquake to observe damages to manholes and to determine whether sewage is flowing in the sewers between manholes. Observations with lights and mirrors will show whether major blockages have occurred. Where sewers are found to be blocked, the location of collapsed pipe sections, broken pieces, or debris causing blockage can be determined through the use of hand-and/or power-rodding devices (064).

Plotting the results of visual and rodding activities on maps and delineating the perimeter of severely damaged areas will show where more detailed damage survey efforts will be necessary. To isolate cracked sections, damaged joints, and other types of damages which do not impede the passage of a rodding device, photographing and televising have been used successfully. Because these sophisticated devices are usually utilized during the later stages of the damage survey, they will be discussed in the next section.

Visual inspection and rodding were used in the initial damage survey in Los Angeles following the San Fernando earthquake of 1971. Visual inspection of 350 miles of sewer lines was accomplished in less than a week, and 60 men were able to check 500,000 feet of sewers with rodding devices in about two weeks (064).

Breaks in the water distribution system, as stated previously, can first be located by telemetered information if the system has been so designed, and by observation of boils or water flow in normally dry areas. Areas of ground cracks, slides, grabens, and differential settlement are most likely to have suffered from pipe failure. A complete survey can be made by observing hydrant pressures and local meters, by pressure testing closed lines section by section, or by using noise/vibration detection systems. The MWDSC plans to use a team of divers to inspect and make emergency repairs to underwater structures.

Sewer lines and manholes may be blocked, resulting in sewage overflowing and running into the streets. Pumps and temporary pipelines can be installed to bypass damaged sections of sewers. Overflows should be contained as much as possible and warnings should be posted. To minimize the disease hazard, chlorine or other disinfectants can be used to treat overflowing or standing sewage in public areas (use hypochlorite solution or "HTH" type powder). In Anchorage, Alaska (1964), trucks with open-top oil drums were assembled to pump sewage as soon as the level began to rise in the manholes; this would be useful only for very small flows.

In areas where sewers are inoperable or could possibly contaminate the water supply, individual homes must be notified that no discharge to the system can be allowed.

It should be noted that in most earthquakes to date, severe damage to water systems has been accompanied by severe damage to sewer systems, obviating an immediate need to carry large volumes of sewage away. As the state of the art improves, it is possible, because of the emphasis on fire-fighting, that water distribution systems will become less vulnerable to earthquakes than sewers. This could result in a situation in which the water system is relatively intact after an earthquake, while the sewer system is heavily damaged. In the future, then, the problem of what to do with very large volumes of contaminated water may become acute.

Damage Assessment

Damage assessment includes a determination of post-earthquake needs for water and sewerage services, the capabilities of the surviving systems, efficient allocation of critical supplies and priority of actions, and the most effective way to restore services (216, 126). While systematically gathering descriptions of the location, nature and severity of individual instances of damage, the control center can initiate the damage assessment (210).

The demand for water will be difficult to estimate, even if reliable information has been compiled on user demands under normal conditions. The need for water before and after an earthquake can vary immensely due to fire-fighting, leaks, evacuation, and so forth. An initial step in assessing water needs would consist of obtaining an estimate of the distribution of the remaining population from local government agencies. It must then be determined whether a sufficient water supply in terms of quality and quantity is available from the utility system. Locations of damage to all facilities should be carefully recorded on maps. Combined with information concerning population distribution, the number of customers affected at each location can be estimated. The availability of services to users with critical needs should be verified through direct communication (209).

A first general assessment of the water system capacity should be followed by a forecast of future water availability. A forecast of water reserves and water flows over the following days or weeks can be facilitated by using worksheets such as those displayed in Figure VIII-1, developed by the California Water Resources Center. These worksheets can be continuously updated as repairs are made. The column entitled "normal operating state" will have been completed prior to the earthquake occurrence.

For wastewater facilities, primary considerations include contamination of domestic water supply by sewage, hazardous conditions associated with damage to waste handling facilities, the availability of interconnected utilities such as power and water that may be essential to waste handling opera-

State of Raw Water

			NORMAL OPERATING STATE		INITIAL EMERGENCY STATE		TIME PERIOD 1		TIME PERIOD 2		TIME PERIOD 3	
			capacity	actual	capacity	actual	estimate	actual	estimate	actual	estimate	actual
a	raw water on hand =h of preceding period	MG										
b	rate at which raw water is received	MGD										
c	rate at which raw water is filtered	MGD										
d	rate at which raw water is sold as such	MGD										
e	rate at which raw water is lost to leaks and evaporation	MGD										
f	net rate of change in raw water reserves = b - c - d - e	MGD										
g	net change in quantity of raw water over period of ____ days = f * no. of days	MG										
h	raw water on hand at end of period = a - g (enter as "a" for next period)	MG										

State of Filtered Water

			NORMAL OPERATING STATE		INITIAL EMERGENCY STATE		TIME PERIOD 1		TIME PERIOD 2		TIME PERIOD 3	
			capacity	actual	capacity	actual	estimate	actual	estimate	actual	estimate	actual
a	filtered water on hand =h of preceding period	MG										
b	rate at which water is filtered	MGD										
c	rate at which filtered water is received from other sources	MGD										
d	rate at which filtered water is delivered to users and other agencies	MGD										
e	rate at which filtered water is lost to leaks and evaporation	MGD										
f	net rate of change in filtered water reserves = b + c - d - e	MGD										
g	net change in quantity of filtered water over period of ___ days = f * number of days	MG										
h	filtered water on hand at end of period = a - g (enter as "a" for next period)	MG										

Figure VIII-1. Sample worksheet for forecasting water reserves and water flows (209).

tions, surviving wastewater-contributing industries, and the type and amount of repairs necessary to restore the operation of critical system components (216).

RESTORING SERVICE

Methods for restoring services will include system reactivation and repair and/or the provision of alternate systems as discussed below.

Water Service

Restoration of water services will generally include one or more of three activities. Immediate demands can be met by trucking water for drinking directly to the consumers. If the water service emergency is predicted to extend over a long period of time, temporary, surface-laid distribution lines can be installed. System reactivation or repair activities can be initiated directly following the earthquake occurrence and can last for weeks or months, depending on the severity of the damages sustained.

In the most extreme case, the raw water source itself will have been depleted through firefighting activities or escape through broken pipelines or damaged reservoirs. Alternate water sources can include local industrial and irrigation supplies and water storage in private properties such as tanks, reservoirs, swimming pools and cisterns. A number of emergency sources are also available within the household, such as hot water heaters (216). If local sources are inadequate, arrangements that have been made for procuring water from neighboring jurisdictions should be executed.

Short-term reactivation and repair - Preliminary efforts can be aimed at reactivating easily recoverable system facilities if their repair will enable the existing system to operate or will be essential to the operation of alternate systems. Cases where system repair largely contributed to water service restoration are highlighted here, although actual repair techniques will generally be described in the next section.

When systems are rendered inoperable only by power shortages, standby power can be provided or portable units brought in. In Managua, for example, the National Power Utility provided electric power to water systems, enabling some service to be restored within nine hours of the earthquake (034). Similarly, one water treatment plant in Alaska was inspected hastily and found to be useable; standby generators were started and the plant resumed operation on the day of the earthquake (078). In Sendai, Japan (Miyagiken-oki earthquake), power resumption and emergency repair work enabled some damaged pumping stations to resume operation on the third day (022). In most instances, water facilities have a high priority in resumption of the power supply from the local power utility. Other experience has included the start-up of emergency wells with diesel engines and electric generators (034).

Examples of short-term system reactivation or repair include:

- Bypass of raw water through treatment plants can be used to supply domestic water provided that means to chlorinate the bypassed water are available (078); note that the safety of this will be a function of raw water quality.
- Miyagiken-oki, Japan (1978): In the Miyagi Profecture, 87,740 families in 55 municipalities were without water. Three days after the earthquake, repairs in storage, treatment, and distribution systems were 78 percent complete; the water supply was totally recovered within 10 days of the earthquake (217).
- Santa Rosa, California (1969): Repair of water main damage was initiated five hours after the earthquake and completed by the next

day (047). Short sections of some pipes were removed and long sleeve flexible couplings installed. Small breaks and holes were secured with repair clamps (218).

- San Fernando Valley, California (1971): Restoration of service to the City of Los Angeles was completed through system repair within 12 days of the earthquake occurrence. Details are presented in the next section.
- Managua, Nicaragua (1972): Repair of twenty-four inch water mains was improvised with segments of ductile iron pipe using cast-iron sleeves sealed with leaded joints, until gaskets and clamps could be obtained (034). This activity required two days, after which partial water service was restored (035).

Trucking of water - Tank trucks have often been employed to provide an immediate water supply under emergency conditions. Two types of trucks that can be used are those with permanently attached water tanks and those which pull detachable water tank trailers. In the former case, allocation of water to individual consumers can be controlled by the truck driver. In the latter case, the water tank trailer can be placed in a particular location and left unattended; the customers are trusted to minimize and allocate withdrawals. The overall quantity consumed can be controlled through the timing of periodic deliveries. Because hoarding or wastage may occur when tanks are left unattended, it may be desirable to have an attendant present in either case (209).

Water for tank truck delivery can be obtained from uncontaminated reservoirs and wells, treatment plants or fire hydrants in unaffected areas.

Chlorination can be provided within the tanker itself (215). Arrangements for procuring the trucks should have been made as part of the emergency preparedness program. Good candidates include milk trucks (219), fire engines (128, 013) and bottling plants (209). Deliveries should be made to central locations such as schools and shopping centers provided that access to these centers is unimpeded.

Tank trucks have been used to deliver water after several major earthquakes. Following the 1971 San Fernando earthquake, a few areas in the City of Los Angeles and most of the City of San Fernando had no normal water supply for several days. About 75 tank trucks were deployed throughout the area, some rented and others on loan from a brewery, soft-drink companies and filtered-water companies. The National Guard supplied 35 small two-wheel tank trailers (500 gallon capacity) to schools. All truck tanks were sterilized, chlorinated and equipped with manifolds that had hose-bib outlets for the drawing of water. The manifolds were assembled from pipe fittings and welded into place (039). Twenty-six trucks used in the City of San Fernando alone, with an estimated average capacity of 4,000 gallons, furnished about 104,000 gallons of water to meet the needs of 14,000 people for a period of 4-12 days after the earthquake (220).

Other instances of water trucking include:

- Managua (1972): 43 tank trucks and trailers from the government of Nicaragua, the U.S. government and a private contractor provided 37,000 gallons of water (220). This was organized on the day after the earthquake. Five thousand filled water cans were flown in from the Panama Canal Zone. Some petroleum product cans were accidently filled; however this was discovered quickly (220).

- Miyagiken-oki (1978): Trucks carrying one-cubic-meter water tanks made 213 deliveries to various locations on the third day after the earthquake (022). Note that the Sendai municipal water utility had a large supply of one-cubic-meter containers on hand for emergency use. An average of 8 to 10 liters per person was distributed during the days of the water shortage (022).
- Tokachi-oki (1968): In Hachinohe, eleven tank trucks supplied water on the day of the earthquake and 31 on the following day. In Gohohe, 2 cubic-meter fire engines supplied water to each house. Water cars or fire engines were also used in Towada, Misawa and Mutsu (013).
- Alaska (1964): Military personnel from Fort Richardson and Elmendorf AFB supplied truckloads of water to Anchorage in hardboard drums with sterile plastic liners and covers. This service lasted for two weeks in some areas (078).

Temporary, surface-laid distribution lines - If the water service emergency is expected to extend over a long period of time, temporary, surface-laid distribution lines can be installed. While affected parties in Anchorage, Alaska (1964) were being supplied with trucked water, plans were made for temporarily providing water through a surface-laid distribution system of 4-inch-diameter irrigation pipes. Lines were fed through short lengths of fire hose connected to fire hydrants. Houses were fed through ordinary 1/2-inch-diameter garden hose tapped from the irrigation pipe; the garden hose was fastened to an outside faucet of the house. The first pipe was put into service five days after the earthquake. Over 14 miles of lines were installed in the area over a period of several weeks (078).

Other types of piping that can be used for surface distribution systems include galvanized iron pipe with threaded couplings and polyethylene pipe, which can be unrolled. Slip joints should not be used for a pressurized system; joints must be of the restrained type. Water main cleaning companies could be a good source of temporary piping.

An efficient combination of above-ground distribution and system repair was used to restore water in the City of San Fernando following the 1971 earthquake. San Fernando did not have sufficient personnel immediately available to cope with its complete loss of water supply. While water was being provided by tank trucks, the City of San Fernando requested assistance from the Corps of Engineers two days after the earthquake. The Corps arranged with contractors to install a temporary above-ground water system to serve part of the city. The system included mains, hydrants, and gardenhose bib outlets for domestic use at each home, as well as 2 1/2-inch outlets at 250-foot intervals for fire protection. Water was obtained from a City of San Fernando reservoir filled from an existing six-inch fire service connection to the Los Angeles water system (039).

To restore service to the remainder of the city, Metropolitan Water District of Southern California forces installed a temporary connection to an 8-inch blowoff on a large pipeline operated by the Calleguas and Las Virgenes water districts two days after the earthquake. It was extended several hundred feet by a 6-inch pipeline connected to a specially fabricated outlet manifold accommodating several firehoses. Over 4,000 feet of firehose was connected from the manifold to fire hydrants in the nearby area (033).

Crews provided by the cities of Pasadena and Burbank were, in the meantime, turning off 5,000 house connections and repairing reservoirs and broken water mains throughout the city. Within one week of the earthquake, about half of the city was being served from the Calleguas conduit, and another 30 percent from the connection to the Los Angeles system (039). A hypochlorinator connected to the turnout from the Calleguas conduit injected a chlorine dose of 20 ppm to disinfect the system (033).

Other methods used for supplying water during past earthquakes include:

- Qir, Iran (1972): By the fourteenth day following the earthquake, a large elevated water tank had been erected, supplied by a fire pump truck which obtained water from a nearby spring. Water was piped to several central outlets in a refugee tent camp (221).
- San Fernando, California (1971): In the first aqueduct conduit at Magazine Canyon, a four-foot high temporary dam was constructed to maintain water in the Maclay High Line supplying the severely damaged Sylmar-Olive View areas (016).

Water quality considerations - It is crucial that a close control be maintained on water quality from all sources. Water tank trucks must be carefully inspected for possible prior contact with non-potable liquids. Even if water is being supplied through pre-existing, supposedly undamaged systems, wastewater escaping from sewers, cesspools and septic tanks can contaminate the water system through leaks and cracks in wells and distribution lines. The above considerations apply also to temporary surface distribution systems.

Water supplies in damaged areas must be sampled immediately and then continuously throughout the emergency period. Pre-arrangements with the local health department or private laboratories will help alleviate the overload on the utility's laboratory facilities. In Los Angeles, for example, water samples were rushed to a laboratory on the day of the earthquake; in anticipation, laboratory personnel had prepared extra culture media and equipment (213). The standard chlorine residual OTA test can be used to measure water supply safety pending the results of bacteriological testing, on the principle that free available chlorine and bacteriological viability cannot coexist for significant lengths of time (126).

If water quality is suspected or discovered to be unacceptable, customers must be warned. The warnings must clearly instruct the consumers to take appropriate action, such as boiling all drinking water. The pre-earthquake emergency plan should include procedures for notifying television, radio, newspapers, and law enforcement vehicles such as trucks and helicopters equipped with loudspeakers. EPA National Interim Primary Drinking Water Regulations state that if a community water system fails to comply with an applicable maximum contaminant level, the public must be notified by publication in a newspaper for three consecutive days, announcements over radio and television stations, and in the first subsequent set of water bills (40 CFR 141).

The degree of water treatment necessary will depend on the judgment of water utility and health department personnel. Maintaining the highest level of free available chlorine consistent with palatability will protect against biological contamination (126). Chlorine doses were maintained, for example, at 50 ppm for several days in Los Angeles (039). Mobile chlorinators can be used to disinfect water as it enters tank trucks or distribution lines.

Excessive turbidity in trunk lines was encountered after the 1971 San Fernando earthquake as a result of slide material from the Lower Van Norman Reservoir dam and bottom muds from the reservoir. On the day of the earthquake, telephone orders were placed for tank-truck delivery of liquid alum to three reservoirs at lower elevations which were receiving water from this reservoir. Each tanker carried 4,000 gallons of liquid alum. The alum was added through pipe headers and nozzles assembled at each reservoir inlet. On the following day, portable 8,000 gallon, plastic-lined swimming pools were purchased and installed at the reservoir inlets, and the trucks discharged alum into the pools (213).

Wastewater Service

Two general types of situations can be encountered in the sewage system after an earthquake. If the water system has been rendered inoperative, or if continued use of sewerage facilities will present such a health hazard that consumers are ordered to refrain from discharging into the system, alternate methods for sewage collection and treatment will have to be provided. If a damaged sewerage system continues to be used, provisions must be made for properly maintaining wastewater routing, treatment, and disposal.

In the former case, the population must be informed of proper waste disposal methods. Individual families can be instructed to use containers to remove wastes and subsequently bury it or take it to a designated disposal site (216). A portable toilet consisting of a seat and disposable bag, normally used by campers, was utilized after the 1964 earthquake in Alaska (078). Portable chemical toilets can be provided for public use at central locations.

Suitable methods for storing or collecting these wastes should be considered in the emergency plan. Tank trucks can be used to haul wastes, or fifty-gallon drums can be distributed at strategic points and subsequently hauled away or pumped out when full (215). In Anchorage, the city's Refuse Division provided specially marked oil drums to selected areas (078).

If the sewerage system is functioning, the considerations will be quite different. Primary considerations here would be to maintain collection capacities, to provide as much treatment as possible, and to discharge it where it will do the least harm.

If sewer lines are blocked, sewage can back up into individual homes. Quick-coupling pipe in conjunction with a pump can be used to bypass blocked pipelines so that sewage flow is maintained. Sewage can be rerouted to a neighboring stormwater system, as was done in one case in Anchorage, Alaska. Sewage can be transported to pre-arranged emergency disposal sites through temporary ditches, natural channels or emergency waste pipelines (216). In Alaska, rerouting of sewer lines, trenching and ditching permitted sewage to flow to tidewater without ponding; open sewage flow was heavily chlorinated. Manhole covers were removed from a broken trunk to enable sewage to overflow into a nearby creek rather than back through the line. The creek was chlorinated to control contamination (078). Note, however, that current EPA policy does not permit sewage overflow from manholes.

The primary objective of treating sewage in an emergency situation is to protect public health. If the treated sewage effluent is normally discharged into a domestic water supply source, such as a river, the maximum amount of treatment possible should be maintained. If no domestic use is made of the receiving waterway, the level of treatment is less critical. Disinfection should be maintained as the minimal treatment in all cases. Increased chlorination may be considered in the absence of other types of sewage treatment, but only after communications with water treatment utilities have confirmed that their chlorine supplies are sufficient. Otherwise, all available chlorine supplies should be allocated to the water utility (216).

Treatment may also be provided at mobile treatment facilities. Health departments and civil defense agencies may have portable chlorinators for emergency use. All potential treatment capabilities should be investigated before the decision is made to discharge untreated sewage into waterways, and this decision should be made only after concurring with health and environmental control department personnel.

REPAIR

Although repair activities are discussed under a separate heading, this does not imply that all system repairs are performed as part of a distinct response phase. As discussed earlier, immediate repair of some facilities may be necessary for protecting life and property, and repairs aimed at restoring services may be performed if they can be accomplished more quickly and efficiently than the implementation of alternate systems. Topics discussed in this section include actual repair techniques used during the previously-discussed response phases, as well as the final process of inspecting and repairing severely damaged systems.

System pipeline repairs are normally routine activities; as malfunctions are reported to the water system, requests for repair are forwarded to repair crews. The system typically functions on a first-come, first-serve basis, as resources are normally adequate to handle most system failures (209). Following an earthquake, repair resources may be insufficient to meet all repair demands. Therefore, strategies must be developed to effectively use available resources. Varying emphasis should be given to protecting the population, quickly restoring normal operations, reducing future vulnerability, and improving efficiency.

Decisions concerning repair priorities must be based on a number of factors, such as the number of consumers affected by each repair activity. The need for water must be determined not only for sustaining life, but for sanitation and necessary industrial and commercial processes. Damaged sewers in the proximity of any water supply system must be repaired first to prevent contamination of the water supply (039). The relative importance of an individual facility and the urgency of returning its use, the length of time required to complete a given repair, and the availability of trained personnel and equipment must be considered.

Maintaining a careful inventory of damages and repairs on maps is very important in the repair phase. These maps are crucial also in organizing the detailed inspection and repair efforts. In addition to showing where severe damage requires intense surveying efforts, they will indicate where water and sewer lines lie in relation to other lifelines, thus facilitating coordinationwith other agencies. Maps can also be used to monitor the progress of repair. In the Sylmar area of the San Fernando Valley, for example, transparent plastic overlays were placed over water district maps, and colored grease pencils were used to record water pipes returned to service, chlorine residuals, and other operating data vital to the repair operation (213).

In many cases, outside assistance will be needed for detailed survey and repair efforts. It should be noted that when outside repair crews assist, many of the utility's senior operating personnel may be required to supervise their work (209).

As systems are being restored to their original capacity, it may be necessary to institute water allowances and rationing. Methods for estimating water requirements for various uses and methods for enforcing rationing programs should be developed in the emergency plan.

A comprehensive description of inspection and repair techniques for all systems is beyond the scope of this discussion. Examples of survey and repair procedures for water distribution and sewer lines are presented below; these have been found to be effective during past earthquakes and may be of assistance in determining necessary resources during emergency planning efforts.

Water Systems

Water supply, storage and distribution system repair activities following selected earthquakes are presented below:

- Managua, Nicaragua (1972): Damage to four 2.5-million gallon, circular, reinforced concrete storage tanks consisted of visible cracks on the walls, shallow cracks on the inner columns, vertical displacement of the bottom slab, and destruction of the outlet and inlet concrete pipes. The repair program was initiated by boring holes at the bottom of the slabs and injecting a cement grout. Visible breaks on the walls were cut and refilled with a non-shrinking concrete material. Small breaks on walls, slabs and columns were repaired by injecting epoxy with a hypodermic syringe and sealing with a brush and roller. Expansion joints were cleansed and filled with sisal fiber, sealed with asphaltic rubber, and applied with epoxy. Tanks were tested by filling with water and observing leaks. This repair effort took 4 months to complete (034).

Repairs to steel tanks, which suffered "elephant foot" compression buckling, were initiated within a month of the earthquake; however, it was several months before they were returned to service (035).

Approximately 400 water-main failures were located and repaired within 10 weeks of the earthquake occurrence (037). Repair work on separated joints was initiated using molten lead, but was later hastened when dresser and gibault joints and repair clamps were obtained through an emergency order. In some areas, potable water was reestablished within three weeks of the earthquake (034). In the destroyed center of the city, where there was no immediate need to restore service, laterals were valved off at both ends and no immediate attempt was made to repair them. Repair of the cast iron pipe failures required realignment of the mains and replacement of bell and spigot ends with short nipples and sleeves. As in repair efforts following earthquakes elsewhere, major transmission lines were repaired quickly; however, it was more difficult and slower to track down smaller breaks and leaking abandoned services (037).

- Alaska (1964): Asbestos cement lines in Anchorage were pressure-tested for leaks at 150 psi. Clean breaks were repaired by installing splicing sleeves over pipe breaks. Before backfilling was completed, lines were tested at 60 psi for joint leaks. Lines were thoroughly flushed and chlorinated.

Following the repair of water lines, a pitometer test was performed at 15 locations to determine the velocity of flow and thereby locate leaks. Excavations were made, and corporation cocks containing pitot tubing were installed. The excavations were backfilled with gage lines terminating above ground for connection with testing equipment. Gage readings were recorded with different valving arrangements and at various times of day and night.

To test the water systems of individual residences in Alaska, a pressure pump was attached to the outside faucet and the gage pressure was pumped up to 50 psi. The buried service line was shut off in the house. The test was then repeated with the service line valve opened in the house and the curb cock closed to locate damage to the buried line between the distribution system and the house. Breaks were repaired when the city distribution line was repaired (078).

- Miyagiken-oki, Japan (1978): Because immediate emergency operations in the city of Sendai were directed at preventing uncontrolled flows from broken water mains, 7,000 services were without water on the following morning. After three days of restoration efforts, 800 customers in newly developed residential districts, where instability of fills and slopes caused extensive pipe damage, were still without water. Over 4,000 breaks to service connections, meters, and domestic piping were reported within 18 days of the earthquake. A total of 650 mandays was required to restore water mains, and an additional 1,100 mandays for repairing facilities on individual premises (022).

- San Fernando Valley, California (1971): Cracks in the unreinforced concrete lining of the First Los Angeles Aqueduct were repaired by chipping the cracks out to a depth of one inch and resealing with mortar. The power penstock, a 96-inch riveted steel pipe serving as one of two means to move aqueduct water into the city of Los Angeles, was placed back into service three days after the earthquake; rivet damage was welded, temporary cribbing was placed under the pipe, and the annular space between piers and pipe, which was as much as six inches in some locations, was repaired. Within one week of the earthquake, heavy fracturing of the First Los Angeles Aqueduct Cascades Channel's unreinforced concrete lining was repaired by chipping and sealing with mortar and by grouting voids beneath the floor with cement (033).

An interesting repair concept was used in Los Angeles, where the water distribution system was threatened from breaks in sewer lines, cesspools, and gas lines. A block-by-block approach of repairing and pressurizing and disinfecting waterlines at lower elevations, gradually forcing water to higher levels, reduced this potential hazard (039). Water imported from outside sources was chlorinated to levels of 100-150 ppm by portable chlorinators mounted on trailers and used to pressurize water mains. Leaks were located and the mains were depressurized and repaired. The highly chlorinated water remained in the main for over an hour and was then flushed into another main (213). Note that normal pipeline disinfection procedures require maintaining 50 ppm available chlorine in the pipe for at least 24 hours (184).

The LADWP worked continuously for a total of 240 crew-days to repair over 1,500 main and service breaks in 110 miles of water lines. Restoration of service was completed within 12 days of the earthquake. Small corrosion holes in steel pipe were repaired with steel screw plugs. In highly corrosive soils, a cathodic protection system was installed to retard further corrosion. Larger holes in steel pipe were repaired with pipe repair clamps, or with wooden plugs covered by pipe repair clamps (033).

Full circle repair clamps were used for circumferential breaks on cast iron water mains; a schematic of a typical clamp is shown in Figure VIII-2. Shattered cast iron mains were replaced with new pipe sections and slip sleeves, while smaller shattered pipe pieces were repaired by slip sleeves or with two full-circle repair clamps (033).

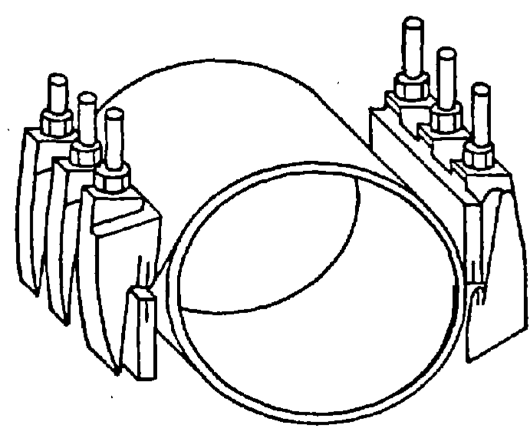


Figure VIII-2. Typical full circle repair clamp (222). (Rockwell International Corporation, Pittsburgh, Pennsylvania).

Joint failures on cast iron mains were recaulked with lead. Joints on some steel mains were repaired with steel butt straps, and bell and spigot joint damages on others were fixed by welding filler rings into the bell. A steel butt strap with lead poured between the pipe and the butt strap was used to repair a joint leak on a cement main. Where bell ends on pipes, gate valves or fittings were broken, lengths of new pipe or a new gate valve or fittings were often required. At some locations, the broken bell was repaired with bell-end repair clamps (033). A typical bell-end repair clamp is shown in Figure VIII-3.

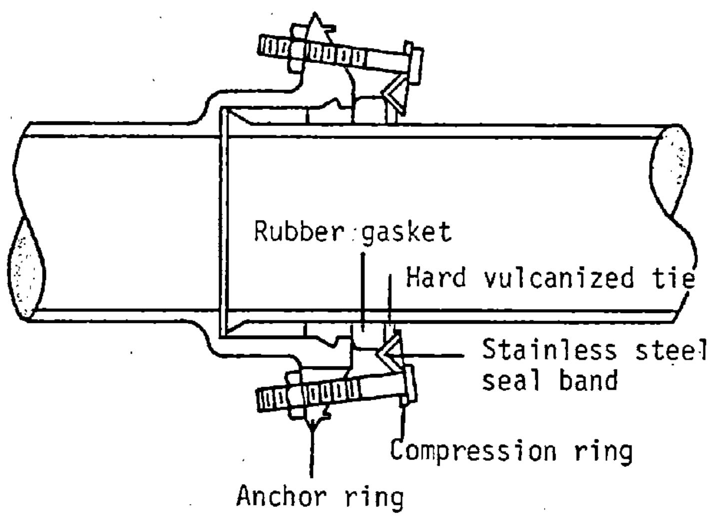


Figure VIII-3. Typical bell joint clamp (223). (M.B. Skinner Products, Bridgeport, Connecticut).

Minor rubber-gasket joint leaks consisting of an unseated gasket were repaired merely by reseating the gasket; pipe repair clamps and slip sleeves were required for more severe joint failures. Welded, riveted and grooved-type coupling joints that were leaking or ruptured were repaired with welded steel butt straps. Separated joints on copper mains were repaired with unions. Fire hydrant laterals were replaced, joints on laterals recaulked, and tees and fire hydrant shutoff gate valves replaced when required (033).

Damaged fittings and tubing on copper services were replaced with new bronze fittings and copper tubing. Ruptured copper service joints were generally repaired with couplings. A Dresser Style 38 coupling, typical of those used, is shown in Figure VIII-4. Leaks on galvanized steel and double cast iron service laterals were often repaired with repair clamps. Deteriorated galvanized services were replaced with copper tubing and bronze fittings (033).

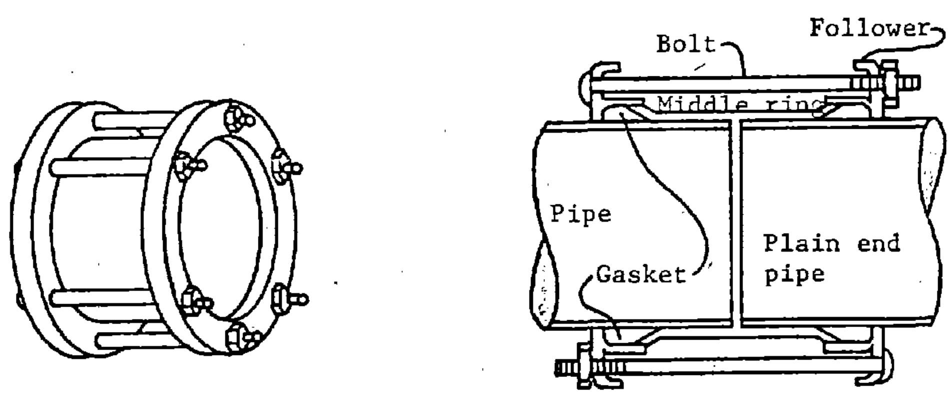


Figure VIII-4. Dresser Style 38 coupling (185). (Dresser Industries, Inc., Bradford, Pennsylvania).

Water leaving the Lower Van Norman Reservoir carried mud, sand, rocks, and large pieces of concrete into the water distribution system. About three weeks after the earthquake, the LADWP initiated a program to flush water mains in an area of about 50 square miles. Some of the transmission and trunk lines were cleaned by cutting the sections out and manually shovelling out the debris. Holes were cut in other mains, followed by high-velocity flushing into flood control channels. In grid systems, fire hydrants were removed and adapters installed on the risers to allow maximum flow. Flushing of the water distribution system was completed within a 4 month period (213).

In the city of San Fernando, more than 17,000 linear feet of cast iron pipe was replaced with locally-available asbestos-cement pipe with neoprene ring joints. This type of pipe was strong enough to withstand the high water pressure. About 10,000 linear feet of thin-walled riveted pipe was also replaced with asbestos-cement pipe. Approximately 1,200 linear feet of concrete-steel cylinder pipe required replacement (033).

Four of five water storage reservoirs in the City of San Fernando were damaged during the earthquake. Three of the four damaged were placed back in service about one month after the earthquake. Horizontal cracks in the lower portion of the walls of one reservoir were planned to be sealed with an epoxy grout (033).

The major impact of the earthquake on wells in the city of San Fernando was contamination resulting from broken sewer lines and septic tanks. Only one of seven wells was damaged severely. Bacteriological contamination was controlled by use of a chlorinator.

Repairs to minor cracks in the pump pad and concrete floor slab in the pumphouse of Well No. 2 consisted of replacing the pump base and sealing it to the well casing. To correct contamination of Well No. 3, nearby broken sewerlines were repaired, and septic tanks were cleaned out and backfilled. A 15-inch steel liner was sealed inside the casing. The original casing was perforated, apparently for the purpose of disinfecting. The ground surrounding the perforations was treated with 10 pounds of concentrated chlorine and 1,000 gallons of water. Eleven cubic yards of grout cement was pumped into the formation under 100 psi and permitted to set for 24 hours. A new pump base was constructed, and the pump was sealed (033).

The State Health Department ordered the city of San Fernando to make many improvements in these wells, and it was many months before all were upgraded to meet requirements. One requirement was that each be equipped with a gas chlorinator and a chlorine-residual recorder (039).

- On June 19, 1975, a large landslide in Ogden, Utah ruptured a 42-inch water main carrying water to a 200,000 m^3 reservoir; the water main was immediately shut off. The Weber County and Utah Offices of Emergency Services cooperated with the Ogden Defense Depot in procuring 2,400 feet of 8-inch steel pipe to use as temporary lines across the landslide. In the middle of each line, a 10-inch long rubberized section was installed for flexibility in the event of another slide (224).

Sewer Systems

The initial investigation of damages to sewer lines using visual inspection and rodding techniques was described in the previous section. These activities will have isolated the most seriously damaged areas quickly and fairly inexpensively. Where damage is severe, television or photography can be used. Photographic inspection of sewers was used successfully in Alaska (1964), although it has more recently been replaced by televising and videotaping techniques.

Televising of sewer lines involves passing a line through a sewer from an upstream to a downstream manhole. A cable for pulling the TV camera is attached to this line, which is pulled back through the sewer to the upstream manhole. The television camera and supporting videotape equipment is attached to the cable at the upstream manhole and then passed through the sewer. The major advantage of television as opposed to photography is that repairs can be initiated immediately in the former case, while the latter requires a separate repair effort after the photographs are developed.

Television cameras and supporting videotape equipment were used to inspect sewers in Alaska, San Fernando, Los Angeles, and Sendai (Japan). In Los Angeles, 50,000 feet of sewers were inspected by one television camera during the first month. Because one camera was inadequate for inspecting the 110 miles of sewers in the most severely damaged area, aid was requested from the Corps of Engineers. The Corps hired a number of private companies to complete the survey of the Los Angeles as well as the San Fernando sewer lines. Within seven months of the earthquake, 110 miles of mainline sewers had been televised. At one time, 11 television crews were operating. The cost ranged between $0.35 and $1.20 per foot of sewer (064).

Significant factors found to require special attention in Los Angeles included the condition of the pipe surface, camera travel speed, logging procedures, taped oral descriptions, and the overall coordination of work. Grease and debris in the sewers rendered some of the video tapes useless, and the process had to be repeated after additional cleaning was performed. Excessive camera speeds also resulted in inconclusive video tape; camera speeds of 2 to 3 fps were found to be optimum. In some cases, reruns were necessary because of poorly written logs. Location controls such as street intersections, house numbers, and sewer stationing from record maps were established, and the geographical location was simultaneously recorded orally and logged. Difficulty arising from different data forms used by each television company suggested that standard forms should be adopted in future efforts (064).

In the City of Los Angeles, emergency repairs and cleaning were performed by city crews during the first few weeks to keep the system as functional as possible. Within 30 days of the earthquake, the Corps of Engineers had reconstructed approximately 3,700 feet of sewers. For the remainder, reconstruction plans were prepared on a priority basis as information from televised inspections became available (064).

In Anchorage, Alaska (1964), eleven contracts were awarded for repair of earthquake damage to sewer lines. This consisted of repairing joint separations, relaying certain portions of lines to grade, realigning pipes, repairing broken joints, repairing manholes, cleaning sewers, and photographing lines that could not previously be photographed. Essential repairs were completed within about 6 months of the earthquake occurrence; some other repairs required separate longer-range projects (078).

B. EMERGENCY PLANNING

The objective of pre-earthquake emergency planning is to develop a plan that will protect lives and property and expedite recovery of water and sewage services after an earthquake event. Thus, emergency planning must be geared toward both hardening a system to minimize potential damage and organizing for a strong and ready response capability.

When planning for an earthquake emergency, a water or sewerage utility must first identify and locate critical system components. Critical components are elements of the system which, if damaged by an earthquake, may render the system partially or completely inoperative. The identification of critical system components thus requires an analysis of the interrelationships between individual system components and overall system performance. Elements determined to be critical to the operation of a system can then be "hardened" or strengthened to resist earthquake induced damages. Because physical strengthening of structures is considered in detail elsewhere in this report, it is discussed only briefly in this chapter.

Because there are no completely earthquake-proof designs for every component of water and sewerage systems, a utility must be prepared for some components to fail during a major earthquake. These components with critical functions should be identified where possible, and alternative approaches should be developed for providing that component's function.

Emergency stockpiles of repair materials and equipment should be maintained, along with an itemized listing of the stockpile inventory. Stockpile facilities should be located away from known faults or other dangerous geological areas, so that they will likely be accessible and hopefully undamaged by an earthquake. Stockpile yards in general should be strategically located within a safe distance at several locations near the most critical components of the system.

An important element of emergency planning is the organization and training of personnel. If personnel have been trained and given assignments prior to an earthquake occurrence, confusion and lag-time in their response to an emergency situation will be reduced. Assignments and checklists should be developed and maintained for checking critical system elements for <u>immediate</u> danger to utility personnel and the general public, and for assessing the effect of damage to these components on overall system functioning.

An emergency plan must also include a system for organizing data reported on damages, repairs, location of repair crews, etc. In doing so, an overall picture of the operating status of the water and sewerage system can be centrally maintained, and emergency assignments, equipment and repairs can be directed to high priority areas. It is essential to establish a strong and reliable communications network and management scheme. Emergency operating centers should be set up to act as clearinghouses for information concerning the status of the system.

In planning for an earthquake emergency, a water and sewerage utility should also develop relationships or mutual-aid agreements with outside groups for assistance. Such groups might include neighboring water and wastewater utilities, federal and state agencies, private contractors, etc.

Finally, the entire emergency plan, perhaps including outside groups, should be tested periodically. Practice drills and trials of all elements of the plan should be exercised based on hypothetical earthquake emergencies. In this way, personnel know their roles, and deficiencies or bottle-necks in the emergency plan can be corrected.

Based on the elements identified above, the following subsections will discuss specific approaches, details and alternatives that may be incorporated into an emergency plan.

CRITICAL COMPONENT IDENTIFICATION AND VULNERABILITY ANALYSIS

The first consideration in emergency planning is to identify the critical components of the water or sewerage system. A critical component is defined as a component whose function is either directly or indirectly required to meet specified performance requirements and levels of operation.

All of the components of the system should be identified and evaluated separately and in relation to the total system by selected performance criteria as defined and discussed in Chapter III of this report. Section C in Chapter V includes a discussion of the available methods of determining the interrelationships of individual components within the system. When an analysis is required for purposes of emergency response planning, a qualitative approach to critical component identification will be adequate in most cases. However, when considering structural additions to harden a system, a qualitative mathematical model approach may be considered.

Once the functional analysis has identified the critical components of the water and sewerage system, the vulnerability of each component to seismic forces must be ascertained where possible. These studies should include power supply, communications, equipment, materials and supplies, personnel, security, and emergency procedures. Under assumed characteristics of a hypothetical earthquake, the water utility should estimate the effects of an earthquake on each component of the system, in particular the critical components. A useful tool in conducting a vulnerability analysis is a tabulated worksheet, similar to the one shown in Figure VIII-5.

NAME OF SYSTEM: Ballant Sewer System ANALYST: ________________

EARTHQUAKE INTENSITY (M.M.): VIII ERICENTER LOCATION: 7 mi. S.W. Downtown Ballant

DAMAGE DESCRIPTION: Cracking of concrete block buildings, houses moved off foundations, moderate pipeline failure, equipment not properly anchored moved out of position.

FACILITY	TYPE OF DAMAGE	EXTENT			STATUS
		Minimal	Moderate	Heavy	
Collection System					
Trunk Lines					
Pump Stations					
Treatment Plant					
- Hydraulic					
- Disinfection					
- Primary					
- Secondary					
Discharge					
Personnel					
Power					
Equipment & Supplies					
Communications					
Emergency Response					

FIGURE VIII-5. Vulnerability Analysis Worksheet (after 225).

As discussed in Chapter V, Section C, a number of researchers have attempted to develop quantitative relationships between seismic motions and induced damage. In many cases, there are little or no quantitative data dealing with specific reactions of individual components to seismic forces. Therefore, subjective judgements of the component's susceptibility to an earthquake will have to be made by utility engineers. Potential damage to water and sewerage system components have been identified in Chapter IV of this report.

An important part of a vulnerability analysis is the consideration of the nature of the surrounding geological environment. This approach would point out system components vulnerable to failure from faulting and soil failures. Critical areas of a water or sewerage system can be located through seismic overlay maps. Seismic overlay maps can be prepared from the results of geological and soils engineering studies. The East Bay Municipal Utility District (EBMUD), for example, has prepared a set of seismic overlay maps to aid in improving the planning data base and design standards. Each set of overlay maps contains the following information (120):

- fault zone
- fault lines with definite or approximate location
- relative direction of movement on each side of fault
- location of known or probable cracks
- upthrown and downthrown sides of faults
- sources of information on each fault
- areas of artificial fill
- former tidal flats
- old concealed stream beds

The seismic overlay maps should also indicate areas susceptible to liquefaction.

These maps can be used in conjunction with maps of the distribution system (locating treatment, pumping and storage facilities) and local topography to give a more complete picture of current conditions to be assessed in the vulnerability analysis. Basic maps that should be developed and maintained by a utility should include:

- topography
- pipes and pipe sizes
- location of valves, intakes/outfalls, pumps, etc.
- tanks
- treatment structures
- pressure zone limits

Components which are located in or near areas which are likely to suffer extreme earth movements can be identified. This information should be included in the vulnerability analysis. A quasi-quantitative determination of the susceptibility of facilities to failure from seismic shaking can be made by comparing the designs of existing facilities to those recommended in Chapters VI and VII. Chapter IV highlights the type of damage to which particular types of facilities are susceptible.

The components identified by the functional and vulnerability analyses to be most critical and susceptible to damage should be clearly recorded in permanent utility records and indicated on utility system maps and plans. These components warrant the attention of the utility as focal points for emergency planning.

SYSTEM STRENGTHENING

System strengthening can be accomplished using two approaches. Proper system planning can mitigate potential seismic damage. Generally, high levels of system redundancy and flexibility will reduce the probability of total system failure. Chapter V, Section B, discusses system design. System components can be strengthened to resist specific earthquake induced failure modes. Equipment can be anchored and stiffened, pipes can be laterally supported, and flexible joints can be installed between connected structural systems that may respond to an earthquake differently. Potentially liquefiable soils can be stabilized, and pipe flexibility can be considered in redesigned fault crossings. Seismic resistant considerations and design criteria are included in Chapters VI and VII.

ESTABLISHING RESPONSE AND REPAIR CAPABILITY

Establishing a strong and reliable internal response and repair capability is one of the most important aspects of earthquake emergency planning. It involves the procurement, control, allocation, distribution and use of essential supplies, equipment, manpower, transportation and other resources. A water and sewerage utility should also be prepared to assist other utilities and jurisdictions if necessary. There are several different factors which must be taken into consideration in developing an emergency response and repair capability. These include:

- personnel organization and training
- procedures for damage inspection, assessment and reporting and monitoring the system
- maps and records
- emergency stockpiles of materials and equipment
- communications equipment and operation

The following discussion will highlight aspects of sound emergency planning related to these areas.

Personnel Organization and Training

Normal daily operation for a water and sewerage utility requires the maintenance of a management staff and repair crew to handle periodic problems within the system. Under normal conditions, damages are phoned into the utility by a customer or reported through routine inspection of the system by maintenance crews. It is then a routine matter for a foreman or supervisor to dispatch available maintenance crews and equipment to assess the damage and complete necessary repairs. However, in the event of an earthquake, when much of the system might be extensively damaged, routine or normal organization channels and maintenance procedures may not suffice. Therefore, the organization and training of both regular and auxilliary personnel in emergency operations and procedures is essential.

The critical components identified through the functional and vulnerability analyses dictate priorities for manpower. An inventory of skills possessed by in-house personnel should be developed, along with staffing requirements for each inspection and servicing of each critical component. By comparison of the two lists, shortages in numbers or skills of personnel can be identified. Pre-arrangements should be made to fill manpower vacancies with auxilliary, voluntary, or other utility and outside agency personnel. Having identified the manpower requirements for each key facility, orderly (and redundant) assignments of personnel to tasks to identify immediate hazards and operational needs should follow (120). Alternates should be designated in case some personnel are unable to reach their assigned posts due to injury, family needs, transportation problems, etc.

An alerting procedure, complete with phone numbers of all personnel and alternate means of communication and assembly, should be prepared and readily available in the event of an earthquake. The list should include primary and alternate assembly points for each personnel.

Key personnel positions of the utility, such as maintenance supervisors, chief operators, chemists, etc. should be organized into a clear line of succession, in order to establish channels of commands and communications. Authority should be given to several individuals to contract for outside assistance before an earthquake takes place to avoid time consuming administrative procedures. Lists of available contractors should be maintained. An example of emergency operations organization for a wastewater treatment plant is shown in Figure VIII-6.

Procedures and Checklists

The utility staff should be responsible for developing and maintaining current emergency operating procedures for mobilizing and employing personnel and resources in varying situations. These procedures include standard methods for damage inspection, assessment, and reporting and monitoring and restoring the system. Examples of such procedures are highlighted below:

- Checklists for water and wastewater plant personnel to evaluate operating conditions which include:
  - power status (primary or emergency)
  - influent/effluent flow conditions (both quantity and quality)
  - tank status
  - damage survey of the entire plant
- Procedures for rapid draining of reservoirs
- Checklists of pre-designated key facilities which should be inspected to determine the situation in their vicinity and their

Figure VIII-6. Emergency Operations Organization: Wastewater Treatment Plant (130).

FACILITY SUPERINTENDENT	MAINTENANCE SUPERVISOR	CHIEF OPERATOR	CHIEF CHEMIST
Upon receipt of emergency condition message, activates appropriate portion of emergency operations plan based on initial alert information. Brings together key personnel to assess severity and outline response actions. Key personnel might include: Maintenance Supervisor, Chief Operator, Chief Chemist, and representatives from organizations providing assistance through mutual aid agreements. The various department heads are responsible for mobilizing their staffs and the facility superintendent should support this effort. Notify State Water Pollution Control Organization of emergency situation, if applicable, and/or request assistance as required. Monitor and support all emergency response actions as required until normal operation is restored. Critique emergency operations plan and upgrade the plan as required. Areas to be reviewed include: response time; adequacy of emergency procedures, equipment, communications and personnel training; process flexibility; and performance of auxiliary personnel and mutual aid agreements.	Mobilize emergency maintenance teams as dictated by nature of emergency. Support emergency operations actions with personnel, equipment and maintenance skills. Coordinate with organizations providing specialized maintenance skills and equipment through mutual aid agreements or contracts. Monitor and support as required all emergency maintenance team actions until normal operation is restored. Critique maintenance aspects of emergency response and provide input to facility superintendent's overall emergency operations critique.	Mobilize emergency operating staff as dictated by nature of emergency. Provide facility superintendent with input concerning operational actions to minimize public health and environmental impact of incident. Montior and support as required all emergency actions involving operators until normal operation is restored. Critique the emergency response as viewed by the plant operator and provide facility superintendent with input to his overall emergency operations critique.	Mobilize laboratory staff and conduct sampling for process control and severity analysis as required. Ensure facility superintendent and chief operator are kept up to date on results of sampling during the emergency. Monitor and support as required activities of laboratory personnel. Critique actions of laboratory personnel during emergency situation and provide input to facility superintendent for his overall critique.

ability to function, including:

- water transmission and distribution lines
- pumping stations
- sources such as wells
- sewage collection systems

- Checklists of pre-designated emergency facilities requiring continuous water and sewer service such as hospitals and nursing homes
- Procedures for determining repair priorities based on type of use, i.e., residential, commercial, industrial
- Procedures for allocation of curtailed water supplies similar to those instituted during drought periods, i.e., rationing. The Metropolitan Washington Council of Governments has a Water Supply Emergency Plan consisting of the following basic elements: (Note: Restrictions are in addition to those previously described.)

  a. Restriction Stage (15% reduction) Limiting residential water consumption limiting use for gardening, swimming pools, car washing, etc.

  b. Emergency Stage 1 (15-20%) Limiting commercial use, i.e., cooling, recreational, restaurants, etc., and prohibition of certain outdoor uses.

  c. Emergency Stage 2 (40-60%) Prohibit non-residential cooling below 80°F and limit commercial use, i.e., laundries, etc.

  d. Emergency Stage 3 (60%+) Commandier all water supplies, no air conditioning in non-residential buildings, no water to industries

- Lists including names, addresses and phone numbers of available contractors that could be retained, equipment that could be rented, and materials that could be purchased, should be maintained. Continuing agreements for services may be drawn up prior to the disaster to avert delays.
- Procedures to disconnect water storage facilities from the system immediately following an earthquake to prevent water loss (126).

This is dependent on the extent of system damage and fire flow demand.

- Procedure to shut down filter plants to conserve raw water (126). This assumes a limited raw water source and is dependent on the extent of system damage and fireflow demand. If filter plants are operated continuously, plant start-up lag time following an earthquake, when fireflow demand may be high, can be eliminated. This should be considered in normal scheduling procedures.

- Check facilities known to contain chemicals or other materials that would pose a hazard if released into the sewer system (215). This procedure can be greatly facilitated if the utility has an industrial waste inventory and monitoring system (perhaps a pretreatment program). Precautions which can be undertaken to minimize potential industrial spills include (130).

  - locate industries on map
  - list hazardous materials which might spill
  - names of phone numbers of responsible industrial officials
  - stockpile necessary neutralizing chemicals
  - install waste monitoring equipment at critical sewer locations
  - inspect monitoring equipment

  An example of an industrial waste monitoring system is shown in Figure VIII-7.

- Establish baselines on water quality levels and procedures for sampling the water supply, analyzing and reporting results under emergency conditions (225).

- Procedures for investigating sewers for damage, which include (226):

  - visual inspection at all manholes to establish the approximate perimeter of the damaged area

Name and Location	Industrial Waste Description	Key Personnel
Industries should be in alphabetical order. Location should include manhole where industrial waste enters municipal system.	List waste by common name, chemical nomenclature, and trade name, if applicable. Also list any other hazardous materials on hand that can potentially enter municipal treatment system and give neutralizing agents, if applicable.	Give names, titles and phone numbers of all key personnel. At least one number should be designated as a 24-hour a day number.
ACME Mfg. Co. - Industrial waste is discharged into manhold at intersection of Main and Pine Streets.	Waste is acidic, pH below 4.0 due to presence of sulfuric acid, H_2SO_4. There exists potential for a spill of concentrated sulfuric acid which can be neutralized with strong basic materials such as lime.	John Doe Plant Manager (phone) Bill Smith Maintenance Supt. (phone) Plant Security Office (phone)* *24-hour number

Figure VIII-7. Example of industrial waste monitoring system.

- rodding of sewers to determine location and extent of collapsed pipe section
- televising of sewers to determine exact location and details of additional damage. When using television inspection of sewers, uniform reporting logs and guidelines for documenting damage locations on video tape should be developed

- Standard forms for reporting damages to water and sewerage systems should be developed.

Developing emergency operations procedures and personnel assignments does not complete this phase of emergency planning. The procedures should be continuously evaluated and tested. Emergency operations exercises based on hypothetical earthquake situations should be conducted periodically. The utility should also emphasize a continuous or on-going training program of new and previously trained staff. This can be accomplished through emergency drills, safety seminars, and monthly newsletters.

Maps and Records

In order to facilitate an expedient recovery of the water or sewerage system, the utility must insure that essential records and documents are preserved and stored in a protected location. The utility's essential records should include the following types of information relating to an earthquake emergency (225, 130, 126):

- detailed engineering plans and specifications on all components in the system
- adequate equipment records:
  - date of purchase
  - equipment manufacturer

- local service representative's name and phone number
- service manuals

- maps of pipeline systems, including locations of all valves, numbers, connections, etc.
- emergency facility and auxiliary personnel
  - names, addresses, telephone numbers, disaster responsibilities, skills, etc.
- emergency sources of supply, availability, means of utilizing, and necessary equipment
- amounts, types, and locations of emergency stockpiled equipment, materials, supplies, and chemicals (including repair items), both belonging to the utility and that available in the area

All records should be kept current and readily available at all levels of operation. Neighboring utilities and local disaster assistance agencies should be informed of the content and location of the latest system records.

Emergency Stockpile Equipment and Materials

Normally, a utility has repair equipment and materials on hand to complete maintenance and repair of the water and sewerage system. However, when an earthquake occurs, the supplies may be insufficient to meet those needed for repair of the system. Emergency stockpiles of repair equipment and materials should therefore be maintained. The equipment and materials which should be stockpiled will include both supplies that are normally used (i.e., repair clamps, pipe, etc.) and emergency supplies such as portable chlorinators, portable storage tanks, etc. Consideration should be given to stockpiling the equipment and materials itemized below:

- pipe locators
- lighting equipment
- pipe (including all sizes and types found in system)
- portable generators
- water distribution manifolds
- auxiliary chlorinators (portable)
- fittings and valves
- repair clamps
- portable pumps (fuel and electric operating units)
- portable tanks (collapsible)
- chemicals for disinfection
  - chlorine
  - hypochlorite powder for main disinfection

NOTE: Stockpiled equipment should be checked periodically to assure that it will be operable when needed.

The following items, while maybe too expensive to stockpile, should be located before the disaster with agreements made for their possible use:

- Construction equipment including backhoes, dump trucks, welders, compressers, etc. Agreements may be made with other municipal departments or local contractors.
- Irrigation pipe and couplings possibly available from local farmers, dealers or pipe cleaning businesses
- Television equipment for sewer inspection
- Pipe leak location equipment
- Portable water treatment units - possibly available from federal or state civil defense organizations
- Tank trucks - available from fire department, milk trucks, bottling works, breweries, etc.
- Portable toilets

This list is not meant to be all inclusive. But it does suggest the type of equipment that should be stockpiled by the utility, or available from other nearby agencies, utilities or outside sources. Emergency stockpile yards should be located away from known faults, liquefaction, or other dangerous geological areas, so that they will be accessible and hopefully undamaged. Alternate traffic routes to and from the stockpile yards should be preplanned, in the event that a roadway is impassible due to debris or bridge collapse.

Inventory Communications Equipment for Emergency Operations

A key element to the execution of any emergency plan is coordination of the response efforts. In planning for an earthquake emergency, a utility must study and coordinate all available means of communications including telephone, two-way radio (both mobile and stationary units), public address systems, and radio and television networks.

Two-way radios should be stocked and distributed for emergency since telephone systems may be damaged. They can be used as standby equipment or alternate systems. The "alternate system" function should be preferred, as it is less likely to sustain damage and is more dependable.

The utility should also preplan the location of command posts, control points, and their alternates, accompanied with an inventory of existing communications equipment. Communications headquarters should be decentralized to assure some communications during and following the disaster. Mobile units and emergency crews should be located in or near areas containing major utility system components.

The Metropolitan Water District of Southern California has held field training exercises to test the response and activation of the District's mobile communications trailer at a field site under a simulated seismic event, assumed to have disrupted two of the District's key base radio stations. The trailer is equipped with a two-band short-wave radio transceiver, commercial radio receiver, commercial telephone and television set. It also has a library of system engineering data and information such as: a microfilm file of engineering drawings, a microfilm reader, a set of hydraulic gradeline drawings, wall maps of the areas and systems, a set of facility operations manuals, and other information important to emergency response procedures. The unit can be operated with commercial power, a trailer-mounted standby generator, or on a limited basis with batteries. Special crews are trained and assigned to this unit. This type of mobile communication unit can prove to be invaluable during an earthquake emergency to relay information to other centers and command posts concerning the status of the system, and dispatch repair crews to priority areas of the system.

In general, any radio unit in the system should have the capability to communicate with any other unit in the system. Local agencies should determine the areawide communication network available for use during an emergency, e.g., those maintained by the U.S. Corps of Engineers, state emergency agencies, police, etc.

The utility should establish communications links with a central command post or Emergency Operating Center (EOC) which has been established for the entire region affected by an earthquake. The regional EOC acts as a clearinghouse for information regarding the situation throughout the region, coordinate regional emergency operations, and is the focal point for obtaining support from mutual-aid agencies (215). The EOC is also used to provide press releases to inform the general public of:

- emergency precautions in the event the water supply has been contaminated from septic seepage or hazardous chemical spills
- the location and availability of emergency water supplies

Emergency planning for communications should also include provisions for using loudspeakers to announce warnings. Preprinted signs indicating directions to establish water supply areas, contaminated water supplies, areas of construction, or other signs to help and warn the public are also useful means of communication.

Communications efficiency, like everything else, should be dry-run tested on a regular basis. Whatever form of communication is used, the guiding principle should be clarity, simplicity, and brevity.

ALTERNATIVE APPROACHES TO MEETING SYSTEM FUNCTIONS

Failure of a water or sewerage system to operate is usually attributable to damage sustained by a critical component between the systems source and the consumer. Often, alternative approaches to supplying the function of the damaged component can be implemented. Utilizing fire truck pumpers to replace damaged pumping stations or employing tanker trucks to haul chlorinated water to distribution points, are examples of alternative approaches or improvisations. Also included in this discussion is an outline of different sources from which aid might be obtained following an earthquake and how a utility pre-arranges agreements for such aid in emergency planning.

Alternate Water Supply and Sewage Disposal

During past earthquake situations, utilizing tanker trucks has been successful in hauling and distributing drinking water. In planning for emergency response following an earthquake, the utility should assess the number of tank trucks available from its own resources and those available from agencies such as the highway department, national guard, federal supplies and private companies (i.e., bottlers, milk plants, breweries, etc.). Arrangements for tank truck supplies from outside sources should be made during the planning phase.

Tank trucks can be filled from a number of different sources, which should be predetermined with alternates prior to an earthquake. These sources might include:

- using fire hoses to fill trucks from fire hydrants
- filling trucks directly from reservoirs
- utilizing a riser attachment to a water source equipped with a manifold to fill several trucks simultaneously

No matter what source the tank trucks have been filled from, the water quality should be determined for potential pathogens. If the water quality is unsatisfactory, the water can be directly chlorinated inside the tank truck. Procedures for direct chlorination should be prepared during the planning stage. Adequate chlorine should be stockpiled for disinfection.

Routes for delivering treated water via tank truck should be carefully mapped out beforehand. Once the routes have been established, tank trucks can deliver water by (126):

- travelling the specified routes, and stopping to fill consumers containers directly from the truck. This is facilitated by equipping trucks with manifolds and spigots.
- travelling to a fixed station near an area of need.
- delivering water to a stationary temporary or permanent storage tank which is equipped to provide distribution to the public.

Other sources of water for drinking and firefighting include:

- equipping fire hydrants with taps to serve local residents directly
- setting up collapsible and portable storage tanks at selected points in the city, which can be filled by tanker trucks.
- pumping from nearby stored water sources such as:
  - swimming pools
  - ponds, lakes, (ocean, if nearby waterfront)
  - industrial water storage system

Damage to a component of a water system often causes a dead end or blockage of the flow of a system. Thus, a utility must supply alternate means of rerouting the flow around the damaged component. Damaged components such as pumping stations or broken sections of pipe can be bypassed by:

- valving off the damaged section and rerouting the flow through the distribution network where possible
- using portable diesel powered emergency pumpers or fire department pumpers
- use of fire hose or quick coupling irrigation pipe

Designs of facilities such as pumping stations should provide for by-passing of the facility by including connections for portable pumps and/or facilities for plugging in a portable generator.

Where distribution piping has been totally disrupted, irrigation piping may be used to temporarily replace the system.

A damaged sewage system may cause the flow of sewage to become impossible. This fact by itself or combined with the lack of water necessary for flushing might make it necessary to prevent the use of household toilets. The utility should be able to distribute and install temporary sanitation collection facilities such as "On-The-Job-Johns" or portable toilets such as those used by campers. During the 1964 Alaskan earthquake, portable camping toilets, consisting of a seat and a disposable bag were found to be of great assistance in providing temporary sanitary facilities. The utility can distribute 50-gallon drums at strategic points to receive household wastes and then haul these drums to a sewage disposal facility.

None of the alternatives just described above are meant to provide long-term substitutions for normal operations of a water or sewerage system. However, if such alternatives are preplanned and organized prior to an earthquake, many operating problems and community inconveniences can be alleviated.

Mutual Aid Programs

While the effects of an earthquake are felt over a large area, heavy damages are usually localized. Therefore, one could expect to find neighboring utilities which might have sustained little or no damage, and thus can afford disaster assistance in terms of both materials and manpower. An important aspect of emergency planning is to develop mutual-aid agreements with neighboring utilities and other agencies for emergency assistance.

Mutual-aid agreements can be arranged with local organizations (i.e., contractors, plumbing concerns, garages, industrial establishments, etc.), neighboring utilities, and local, state, and federal agencies (i.e., civil defense, national guard, U.S. Corps. of Engineers, police and fire departments) for disaster assistance. Outside agencies cooperating in mutual aid agreements should be familiar with the water and sewerage system of concern. A utility should provide such agencies with up-to-date maps, records and engineering schematics of the system. This will enable an outside agency to preplan where assistance might be needed.

Many utilities of California have entered into mutual-aid agreements with neighboring utilities and state disaster relief organizations. For example, the Metropolitan Water District of Southern California has explored mutual aid cooperation with the State Office of Emergency Services and other utilities, including periodic review and "dry-run" testing of communications, review of repair supply inventories, and inventory of rolling stock needed to assure continuance of water supply. Frequent interpersonal contact exists between District operations personnel and representatives from other emergency and utility agencies. Mutual aid ranges from the use of stockpiled quick-coupling pipe to locating water tank trucks which can be used in setting up emergency water points. Facilities for emergency interconnections with neighboring utilities have been installed at the District's southern boundary on a shared-cost basis and are being developed in cooperation with an adjacent water district to the northeast. The District has arrangements with contractors to make available crewmen and repair equipment during an emergency (120).

A utility might also make arrangements with such organizations as the Associated General Contractors of America. The Association has developed "Plan Bulldozer" to make available heavy construction equipment, manned and ready to operate, to public agencies in the event of an earthquake.

Utilities and agencies entering into mutual-aid agreements should establish a joint emergency operating center. Emergency operating centers can act as a clearinghouse of information concerning the status of the system and coordinate operations; serve as a focal point for obtaining outside aid from state and federal agencies; and establish communications systems which link to state and federal communications systems.

Community volunteers are a resource of manpower which should not be overlooked in emergency planning. Given proper training to respond to earthquake emergency procedures, community volunteers can participate in clean-up crews, emergency water supply crews, firefighting crews, etc.

In order for utilities to assess the resources of disaster assistance available, a method for evaluating regional agencies and neighboring utilities for mutual-aid assistance should be established. The sample mutual aid information fact sheet shown in Figure VIII-8 represents an example of a tabular method to complete such evaluation. Information obtained through an evaluation of mutual-aid resources can serve as a permanent record of how the utility can obtain assistance.

SAMPLE MUTUAL AID INFORMATION FACT SHEET

NAME	DESCRIPTION OF ASSISTANCE	COORDINATION INFORMATION
Public Works Department	Department of Parks maintains 1,000 feet of 6 inch quick coupling aluminum pipe that is available to assist treatment system during emergencies.	To obtain pipe, contact Dept. of Parks (Phone) during normal working hours or call city switchboard (Phone) after normal working hours.
City Water Department	Water Department maintains 2 portable chlorinators which can be used for emergencies within the wastewater treatment system.	Contact Water Department Supt. (Phone) or operator on duty at main filter plant (Phone).
ABC Construction Company	4 tractor mounted back-hoes are available on a 24-hour basis.	Contact company main office (Phone) or after hours call John Doe, Equipment Foreman (Phone).
ACME Welding Company	Machine shop facilities and a portable welding machine are available on a 24-hour basis.	Call: (Phone) Office (Phone) Home (Phone) Home

Figure VIII-8. Sample mutual aid information fact sheet (130).

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