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SEISMIC DESIGN CRITERIA FOR LIFELINES

by

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June 1997

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The Navy has numerous bases located in seismically active regions throughout the world. Safe and effective structural design of waterfront facilities requires calculating the expected site specific ground motion and determining the response of these complex structures to the induced loading. The Navy's problem is further complicated by the presence of soft saturated marginal soils which can significantly amplify the levels of seismic shaking and liquefy as evidenced by recent earethquake damage. Lifelines are those key public works and utility systems which support the operation of a Navy base. They include electric power, gas and liquid fuels, telecommunications, transportation, port facilities, and water supply and sewers. Safe effective seismic design consist of three components - establishment of performance goals, specification of the earthquake loading, and given that loading, definition of the expected acceptable structural reponse limits. This document gives criteria for the seismic design of lifelines and contains supporting technical commentary.

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Executive Summary

The Navy has numerous bases located in seismically active regions throughout the world. Safe and effective structural design of waterfront facilities requires calculating the expected site specific ground motion and determining the response of these complex structures to the induced loading. The Navy's problem is further complicated by the presence of soft saturated marginal soils which can significantly amplify the levels of seismic shaking and liquefy as evidenced by recent earthquake damage. Lifelines are those key public works and utility systems which support the operation of a Navy base. They include electric power, gas and liquid fuels, telecommunications, transportation, port facilities, and water supply and sewers. Safe effective seismic design consists of three components - establishment of performance goals, specification of the earthquake loading, and given that loading, definition of the expected acceptable structural response limits. This document gives criteria for the seismic design of lifelines and contains supporting technical commentary.

Both Navy and civilian codes have made distinctions between ordinary and essential construction. Generally, essential construction is expected to be operational after an earthquake. Facilities are deemed as essential by virtue of their need after an earthquake such as a hospital, fire station, or emergency recovery center. Navy facilities may also be deemed essential by their mission requirement in support of national defense, such as a communication station. Utility systems are in general deemed as essential based on the needs of fleet operability. The decision to declare a structure as essential is to be made by the user in conjunction with the Naval Facility Engineering Command design agent.

The seismic design of a facility must first focus on the mission of that facility. When evaluating the requirements of an essential facility which must function after an earthquake, the facility must be viewed as a total system including the structural shell, mechanical requirements, electrical requirements, communications requirements, water and sewer, etc. Each of these systems must function so that the mission of the facility may be accomplished. It is not adequate for the structure to survive if the facility is out of service for an unacceptable period from an interruption of any lifeline link.

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INTRODUCTION

Lifelines are key public works and utility systems which are vital to the operation of a Navy base. They include electric power, gas and liquid fuels, telecommunications, transportation, port facilities, and water supply and sewers. Safe effective seismic design consists of establishment of performance goals, specification of the earthquake loading, and given that loading, definition of the expected acceptable structural response limits.

Both Navy and civilian codes have made distinctions between ordinary and essential construction. Generally, essential construction is expected to be operational after an earthquake. Facilities are deemed as essential by virtue of their need after an earthquake such as a hospital, fire station, or emergency recovery center. Navy facilities may also be deemed essential by their mission requirement in support of national defense, such as a communication station. Utilities are in general deemed as essential based on the needs of fleet operability.

The decision to declare a structure as essential is to be made by the user in conjunction with the Naval Facility Engineering Command design agent.

When considering a facility supporting an essential function, it is critical that the facility be considered as a system. It is not sufficient to consider a facility simply as a building structure, but rather it is required to consider all the elements required to accomplish the mission to be accomplished in that structure. This usually includes requirements for electrical power, mechanical systems, water and sewer, communications, road access etc.

LIFELINE PERFORMANCE OBJECTIVES

The following performance objectives are presented herein and are proposed for Navy use. They are based on mandates of public law and extensions of current Navy criteria.

Ordinary Construction / Ordinary Lifelines - Lifeline service associated with construction categorized as "ordinary" shall be designed with the same levels of service. In general ordinary construction is expected to

- Resist a minor level of ground motion without damage;
- Resist a moderate level of ground motion without structural damage, but possibly experience some nonstructural damage;
- Resist a major level of earthquake (10 percent probability of exceedance in 50 years) ground motion without collapse, but with structural as well as nonstructural damage.

Essential Construction / Essential Lifelines - Life line service associated with construction categorized as "essential" shall be designed with the same levels of service. In general essential construction is expected to:

- Resist the maximum probable earthquake likely to occur one or more times during the life of the structure (50 percent probability of exceedance in 50 years) with minor damage without loss of function and the structural system to remain essentially linear.
- Resist the maximum theoretical earthquake with a low probability of being exceeded during the life of the structure (10 percent probability of exceedance in 100 years) without catastrophic failure and a repairable level of damage.

Hazardous Materials/Lifelines - Lifeline service associated with construction categorized as containing "hazardous materials" shall be designed with the same levels of service. In general hazardous material containment construction is expected to:

- Conform with criteria for essential construction
- Resist pollution and release of hazardous materials for an extreme event (10 percent probability of exceedance in 250 years)

Provision for tanks and pipelines containing hazardous materials are discussed further below.

Note the application of Hazardous Materials requirements for extreme event design is not meant to apply to facilities storing small quantities such as in a laboratory.

SEISMIC LOADS FOR LIFELINE DESIGN

The following is based on current criteria and an extension of existing mandates logically applied to analogous situations.

Design Earthquakes

The following criteria are based on current Navy criteria and public law. The Navy lifeline systems shall be designed to resist the loading produced as follows:

- **Ordinary category of construction on average seismicity sites**
  For sites of average seismicity, use code provisions contained in NEHRP, UBC and NAVFAC P355 , which are based on an earthquake with an approximate 10 percent chance of exceedance in 50 years.

- **High seismicity or essential category of construction**
  Sites of high seismicity controlled by local faulting where general code provisions do not account for local hazard potential, or where the lifeline is deemed important and essential shall use a two-earthquake procedure with Level 1 earthquake having a 50 percent probability of exceedance in 50 years and a Level 2 earthquake having a 10 percent probability of exceedance in 100 years based on a local site seismicity study. Values less than code are not be permitted.

- **Construction containing polluting or hazardous material**

An Level 3 earthquake having a 10 percent probability of exceedance in 250 years exposure shall be used.

As part of this criteria:

- the determination of the design earthquake shall be performed using techniques described in criteria for ground motion for essential structures

In addition to seismic ground motion there are additional hazards which must be considered:

- Fault movement and ground displacement
- Liquefaction and associated lateral spreading, settlement flow slides, loss of support and buoyancy of buried tanks.
- Landslides
- Tsunamis

Modification to Design Ground Motion

The ground motions used in design of lifelines may differ from the motions used in conventional building design since the seismic motion on the lifeline may be substantially different than that associated with free-field ground motion. For lifeline component elements located within a structure, the component design loading can be substantially amplified by the response of the structure. In such cases the motion to be used for design of the component must be the local seismic motion transmitted by the structure to the component.

LIQUEFACTION AND LIFELINES

Design of structures shall include provisions to evaluate and resist liquefaction of the foundation and account for expected potential settlements and lateral spread deformation. A Navy criteria document for liquefaction has been prepared separately. Liquefaction is the single greatest cause of damage at the waterfront, especially in wharves, quaywalls and retaining structures. Special care must be given to buried pipelines in areas subject to liquefaction to preclude breaks resulting in release of hazardous materials. The most important element in seismic design of pipelines is proper siting. It is imperative to avoid areas of landslide and lateral spread.

The presence of any potentially liquefiable materials in foundation or backfill areas shall be fully analyzed and expected settlements computed. Specific attention shall be paid to the acceptability of the amount of settlements. Since liquefaction is a major damage mechanism at the waterfront, remediation is a mandatory requirement where the risk of a pipeline break or tank failure is shown by computation to be possible and hazardous materials would be expected to be released.

WATER, GAS AND LIQUID FUEL LIFELINES

Pipelines

Pipelines must be designed to resist the expected earthquake induced deformations and stresses. Generally permissible tensile strains are on the order of 1 to 2 percent for modern steel pipe. To accommodate differential motion between pipelines and storage tanks it is recommended that a length of pipeline greater than 15 pipe diameters extend radially from the tank before allowing bends and anchorage and that subsequent segments be of length not less than 15 diameters.

Flexible couplings shall be used on long pipelines. In general pipes should not be fastened to differentially moving components; rather, a pipe should move with the support structure without additional stress. Unbraced systems are subject to unpredictable sway whose amplitude is based on the system fundamental frequency, damping and amplitude of excitation. For piping internal to a structure, bracing should be used for system components.

- No section of pipe in Zone 2, 3 or 4 shall be held fixed while an adjoining section is free to move, without provisions being made to relieve strains resulting from differential movement unless the pipe is shown to have sufficient stress capacity.
- When secondary or standby gas supply systems cannot be justified for a site, gas distribution networks for buildings in Zones 2, 3 and 4 housing essential functions dependent upon gas shall include an above ground valved and capped stub. Provision shall be made for attachment of a portable, commercial-sized gas cylinder system to this stub.
- For essential facilities in Seismic Zones 3 and 4, an earthquake activated gas shutoff valve shall be provided. If an earthquake activated shut-off valve presents the possibility of disrupted service in the buildings where the fire hazard is small, manually operated valves shall be installed.
- Buildings housing essential functions shall be provided with two or more water service lines connected to separate sections of the supply grid to minimize loss of service. Service shall be interconnected within the building by check valves to prevent backflow.
- Flexible connections shall be used between valves and lines for valve installation on pipes 3 inches or larger in diameter.
- Flexibility shall be provided by use of flexible joints or couplings on a buried pipe passing through different soils with widely different degrees of consolidation immediately adjacent to both sides of the surface separating the different soils.
- Flexibility shall be provided by use of flexible joints or couplings at all points that can be considered to act as anchors and at all points of abrupt change in direction and at all tees.
- NAVFAC P355 paragraph 12.-7 specifies restraints for critical piping in essential facilities.

Piping containing hazardous materials shall contain numerous valves and check valves to minimize release of materials if there is a break. A secondary containment system should be incorporated where feasible. When piping is connected to equipment or tanks, use of braided flexible hoses is preferable to bellow-type flexible connectors since the latter has been noted to fail from metal fatigue. Welded joints are preferable to threaded or flanged joints. If flanged joints can not be avoided the use of self-energizing or spiral wound gaskets can allow a bolt to relax while continuing to provide a seal, Association of Bay Area Governments (1990). Seismic shutoff valves should be used where necessary to control a system or process.

Tanks

All tanks must be anchored. A pattern of well distributed anchor bolts works best compared with fewer larger bolts. A maintenance program is required to inspect the condition of the anchor bolts. Bolts showing corrosion must be replaced. Vertical motion can cause local tensile membrane deformation, elephant foot bulging, at the base of the tank. Tank venting is important to restrict implosion.

- Typically anchor bolts for new construction are designed with a safety factor of 4; a value of 3.0 is used for evaluation of existing anchors. Provisions must be made to evaluate the effect of corrosion in reducing the strength of existing construction.

To achieve the required system performance and satisfy regulations, additional hazardous material containment systems are usually used as a backup. Containment systems are composed of either a singular system or a dual system as mandated by public law as discussed in the Commentary. A singular system provides only a single structural element system for material containment. Singular systems are restricted to small systems of less than 660 gallons such that a failure shall not produce catastrophic damage. A dual system is composed of a primary containment structure and a secondary containment system which shall function should the primary system be damaged. Containment systems open to rain will need to be drained.

Design of tanks shall utilize the procedures discussed below.

- Tanks shall be designed against sliding and uplift and be fully anchored.

- Essential tanks shall be designed to resist Level 2 earthquakes (10 percent probability of exceedance in 100 years) using response spectra and the API 650 procedures.

- For both ordinary and essential tanks, a requirement exists to prevent uncontrolled loss of contents and pollution of the environment for an event with a 10 percent probability of exceedance in 250 years. This is discussed below in the section Hazardous Materials Containment.

Such spill containment requirements may be met by provision of a containment system. Singular systems must be designed so that the structure itself provides the margin of safety to preclude release of materials. Dual systems may be evaluated on the basis of total system performance allowing for the presence of the secondary confinement, such that any release from the primary containment is confined within the secondary containment. The secondary containment must function at such a level so as not to permit an unacceptable release of materials. This requirement will be discussed below.

Failure of pipe to tank connections is common when there is insufficient flexibility to accommodate differential motion between the tank and pipe network. This can be prevented by having the first pipe anchor point at a sufficient distance (15 pipe diameters minimum) from the edge of the tank and the pipe oriented in a radial direction away from the tank. Additionally stairways should not be attached to both the foundation and the tank wall.

API 650 states that piping attached to the tank bottom that is not free to move vertically shall be placed a radial distance from the shell/bottom connection of 12 inches greater than the uplift length predicted by the API 650 uplift model. The API model may under predict the uplift so a value of twice the API shall be used.

Design of New Tanks

The procedures described in American Petroleum Institute Standard 650 (1993 with updates through 1996) shall be used as modified and updated so as not to produce lower loads than what would be required by the Uniform Building Code, FEMA 223A Sec 3.1.3, and NAVFAC P355.

For essential tanks, response spectra values shall be substituted for equation values. The procedure considers that the loading consists of components at the tank fundamental frequency and also components at the sloshing frequency. Response spectra values based on a tank period shall be substituted for ZIC_1. Additionally, sloshing period values shall be substituted for ZIC_2. Tank wall stresses are computed from overturning moments and compared with allowable values. The user shall consider the amount of tank freeboard for sloshing. Failure to provide for sloshing could damage the roof if the tank is completely full. Provisions are included to allow for local site conditions. A 2 percent damped curve is recommended for design of the structure, and a 0.5 percent damped curve is recommended for sloshing of the liquid.

Evaluation of Existing Tanks

Existing tanks shall be evaluated using the procedures for new tank design; however, when an existing tank is found to be deficient it shall be checked using the procedures described by Manos (1986). Since the new tank design procedures are conservative, an existing tank may be considered as acceptable if it meets the provisions in Manos (1986) and has a lateral acceleration capacity in excess of demand.

HAZARDOUS MATERIALS CONTAINMENT

Performance Goal

This section of the criteria is intended to address the seismic design of Navy industrial/hazardous facilities, tanks and pipelines which contain hazardous materials. This criteria is intended to produce a level of design such that there is a high probability the facilities and components will perform at satisfactory levels and prevent a release of hazardous material throughout their design life. Specifically for industrial/hazardous facilities, tanks and pipelines located in areas of high seismicity, such as Uniform Building Code designated Seismic Zones 3 and 4, structures shall be designed:

- To meet all of the provisions for tanks given above.

- To resist major earthquakes, Level 3, which are considered as infrequent rare events without uncontrolled release of hazardous materials.

Design Earthquakes

The Navy industrial/hazardous facilities, tanks and pipelines shall be designed to resist the loading produced as follows:

- For sites of average seismicity, use NEHRP, UBC and NAVFAC P355 Chapter 13 provisions, which establishes the earthquake at a nominal 10 percent chance of exceedance in 50 years.

- For sites of high seismicity controlled by local faulting where general code provisions do not account for local hazard potential, or where the tank is deemed important and essential use a Level 2 earthquake with a 10 percent probability of exceedance in 100 years exposure and increase Zone Factor coefficient per response spectra techniques based on a local site seismicity study. Values less than FEMA 223A Sec 3.1.3 and NAVFAC P355 are not be permitted.

- Use a Level 3 earthquake with a 10 percent probability of exceedance in 250 years exposure for spill prevention.

Industrial/Hazardous Tanks and Pipelines Response At Design Loading Levels

Containment systems shall be composed of either a singular system or a dual system as mandated by public law discussed in the Commentary. A singular system provides only a single structural element system for material containment. Singular systems are restricted to small systems such that a failure will not produce catastrophic damage. A dual system is composed of a primary containment structure and a secondary containment system which will function should the primary system be damaged.

The structural response of the Navy industrial/hazardous facilities, tanks and pipelines under the design earthquake levels shall be:

- Meet all requirements for nonhazardous material tanks

For a Level 3 earthquake, controlled inelastic behavior with maximum ductility factors to preclude release of hazardous materials. Singular systems must be designed so that the structure itself provides the margin of safety to preclude release of materials. Dual systems may be evaluated on the basis of total system performance allowing for the presence of the secondary confinement, such that any release from the primary containment is confined within the secondary containment. The secondary containment must function at such a level so as not to permit an unacceptable release of materials.

Design of structures shall include provisions to evaluate and resist liquefaction of the foundation and account for expected potential settlements and lateral spread deformation. Special care will be given to buried pipelines in areas subject to liquefaction to preclude breaks resulting in release of hazardous materials. The most important element in seismic design of pipelines is proper siting. It is imperative to avoid areas of landslide and lateral spread The presence of any potentially liquefiable materials in foundation or backfill areas shall be fully analyzed and expected settlements computed. Specific attention shall be paid to the acceptability of the amount of settlements. Since liquefaction is a major damage mechanism at the waterfront, remediation is a mandatory requirement where the risk of a pipeline break or tank failure is shown by computation to be possible and hazardous materials would be expected to be released.

UTILITIES ON PIERS

Piers may contain pipelines for freshwater, saltwater, steam, compressed air, waste oil, sewer, fuels, as well as electrical power and communication lines. Ship demands dictate the configuration. In general design of these lines follows the general provisions discussed herein. It is essential that the lines be attached to the supporting structure with sufficient rigidity that the lines are restrained against independent movement. Attachments to a pier may be analyzed as simple two-degree-of freedom systems as discussed in NAVFAC P355, Chapter 12. Resonance amplification can occur when the natural period of the supported pipe is close to the fundamental period of the pier structure. Flexible connections/sections shall be used to bridge across expansion joints or other locations where needed. All piping and utility lines on a pier shall be designed as essential construction. Specifically, the provisions of NAVFAC P355 Section 12-7d shall be used.

ELECTRIC POWER

Criteria for electrical power lifelines focuses on providing adequate anchorage. All transformers on poles or platforms shall be anchored against overturning or sliding. All equipment shall be anchored as required. Equipment deemed as of ordinary importance shall use lateral force requirements based on provisions of the 1994 Uniform Building Code and NAVFAC P355. Equipment deemed as essential shall have the lateral force requirement computed based on local site conditions using peak ground acceleration for essential level facilities (10 percent exceedance in 100 years) and a response spectra. In any case lateral forces shall not be less than Code provisions with an importance factor for essential structures/components . This resulting force shall be used as a substitute for Code forces and all remaining Code provisions will apply.

Snubbers by definition are restraints with an air gap. Such anchorages can amplify seismic motion by having equipment bang against restraints. Use of resilient grommets or molded epoxy grouting can eliminate the air gap and reduce or avoid hard surface contact. The snubber and the connection of the snubber to the equipment and structure must have sufficient strength to transmit the inertial forces. Seismic isolation can be an effective technique for reducing loading on floor mounted equipment. Seismic isolation can be used in addition to snubbers or can be made a part of the snubber. Proper anchorage capacity including both horizontal shear and overturning uplift is required and a wedge anchor is recommended. Poured in place anchors are often not feasible for snubber tie-down since equipment location is variable and may not be defined specifically. Snubbers must be omnidirectional with at least a 3/8 inch resilient collar; at least 4 snubbers must be used and all snubbers must be rated. Adequate accommodation of differential motion among components must be provided to prevent failure of items like ceramic insulators etc. Adequate cable slack or break away connections must be used.

TELECOMMUNICATIONS LIFELINES

Telecommunications encompasses conventional telephone requirements, communication transmitters and receivers in conjunction with airfield operations, and military requirements in support of operational readiness. Systems include buildings with electronic components, large computers on raised floors, cables in racks, batteries on shelves, conventional telephone lines, microwave antenna, satellite communications, towers for antenna, etc. As can be noted, there is a diversity of requirements; yet, all of the elements have a commonality. First the equipment must be rugged enough to withstand the shaking. The IEEE has established fragility requirements for some equipment found in nuclear power plants. Some equipment have fragility data. Second the equipment must be attached in a manner to prevent damage. Attachment can be made by rigidly securing the item against overturning and sliding or where the equipment is delicate it may be mounted on isolators to reduce transmitted motions. A variation of both approaches consists of leaving a large piece of equipment free to slide within restrained limits to prevent overturning.

Traditional damage has included overturning of cabinet mounted electronics, failures of battery racks, failures of suspended ceilings, rupture of piping and water damage to equipment, rupture of cables connecting equipment which became dislodged, weld failures, and inadequate sizing of restraints. Design rules must consider the inertia force of an object in overturning and sliding. Elements attached to the structure must consider the relative displacement between anchorage points. Flexible supports must consider resonance points when the period of vibration of the flexible mount is the same as that of the structure; stiffening the mount can eliminate resonance.

For best seismic resistance, raised floor pedestals should be considered as cantilever columns and anchored into the supporting concrete slab. Diagonal bracing is available and can be used for additional restraint. The stringers should be firmly attached to the pedestals for added bracing.

Appendix

Commentary To Seismic Design Criteria

For

Lifelines

INTRODUCTION

Lifelines are key public works and utility systems which are vital to the operation of a Navy base. They include electric power, gas and liquid fuels, telecommunications, transportation and water supply and sewers. The following pages explain the development of the proposed design criteria and give current Navy procedures and all relevant codes. Observed damage from previous earthquakes was analyzed to develop failure modes, from which a design criteria was produced.

As the year 2000 approaches, the US Navy is the dominant seapower capable of significant projection of force; however, not all threats are from hostile combat. Today, the current potential for long term disruption of Fleet operations is probably greater from seismic hazards than combat. The 1980s began a period of increased seismicity in the San Francisco Bay Area. The probability of a magnitude 7 or larger earthquake in the Bay Area is 67% in the next 30 years (or 33% in the next 10 years) USGS, (1990). The probability of a magnitude 7.5 or larger earthquake on the southern San Andreas fault is 60-70% in the next 30 years or 20-30% in the next 10 years. There is a 50% chance of a magnitude 6.5 to 7 earthquake on the San Jacinto fault in southern California in the next 30 years or 20% in the next 10 years, USGS, (1988). While lifeline disruption and hazardous materials problems can occur with smaller earthquakes of magnitude 5 to 6, the number of these problems, and the size of the area affected, increases dramatically for larger earthquakes and for earthquakes in urban areas. The waterfront location on soft marginal soils makes the Navy especially vulnerable. The entire Pacific rim is at risk, covering all PACFLT locations. Additionally parts of the East coast and Europe have the potential for large earthquakes. The Navy has lost over $275 million in damage from the 1989 Loma Prieta and 1993 Guam earthquakes. Executive Order 12699 and 12941 mandate that the federal government develop appropriate technology for safe facility design and evaluate the current vulnerability of government facilities.

Potential problems facing the Navy after an earthquake are building structural failures, damage to waterfront retaining structures, tank failures, crane failures, utilities disruption and hazardous materials spills. Typical lifeline problems involve above ground and underground pipeline breaks from soil movement, collapse of pipelines caused by failed supports, shifting of tanks on their foundations, and buckling of tanks. Related factors which add to the complexity of recovery are the dislodging of asbestos or encapsulated asbestos insulation; industrial equipment damage caused by sliding or overturning, or internal failures; falling containers of hazardous materials which may rupture and impede recovery. Other factors can complicate the ability to respond to these releases, including: lack of water for washing down spills, disruption in communication, closure of roads, and lack of transportation access routes.

Safe effective seismic design consists of three elements - establishment of performance goals, specification of the earthquake loading, and given that loading, definition of the expected acceptable structural response limits. Apart from buildings, the Navy has limited current seismic load-performance criteria. For example, standards do not exist for defining acceptable factors of safety against liquefaction in soil, a major cause of high damage at the waterfront. While we have the technology to perform the structural and geotechnical analysis to evaluate the occurrence of liquefaction and the response of a structure, we lack the guidance standards which tell us what constitutes acceptable behavior under some associated load level.

The current work is a limited program to develop a general guide specification for seismic loading and performance in areas where deficiencies exist. A requirement exists for standards in the following areas: dry-docks, retaining walls, and hazardous facilities/ industrial facilities. This commentary addresses the area of lifelines. Previous criteria documents covered piers and wharves, ground motion, and liquefaction. Text shown in italics is formally a part of the proposed criteria specification for lifelines.

It is important to understand that a complete design standard is composed of three major parts:

1. Development of a set of performance goals defining levels of operation required after earthquakes of various sizes.

2. Specification of a set of earthquake loadings either by deterministic methods (specified as perhaps maximum event) or probabilistic methods (probability and exposure time such as 10% probability of exceedance in 50 years or a nominal 500-year return time event), and

3. Determination of the response of the structure at the specified loads such as drift limits or ductility limits which will ensure damage is limited to meet the expected structure performance.

Thus, full design criteria includes definition of :

1. Performance objectives,
2. Specification of loads,
3. Definition of component damage mechanisms all elements of the structure,
4. Evaluation of all possible failure modes of the global structure,
5. Incorporation of current design and analysis practice (static or dynamic),
6. Development of allowable response limits such as stresses, ductilities, and drifts to control element damage and structure performance,
7. Evaluation of economics of design,
8. Understanding of the structure reliability

In general, a reliability analysis evaluates the loading conditions with their measure of uncertainty and the composition of the structure in terms of material properties, structural member sections used, the uncertainties in materials and construction etc. From the quantification of uncertainty one can calculate the distribution of possible performance outcomes.

The Navy base functions as a small, self-contained city. It has water and sewage distribution systems which employ pipelines, tanks, pumping stations and control centers. Fire fighting may be a part of this system or a separate system. Central steam systems are used for used for building heating and shipboard power. Fuel systems employ networks of tanks, pipelines and pumping stations. Industrial process buildings related to ship or aircraft maintenance often house tanks, pipelines and equipment involving hazardous materials. Each of these systems presents a potential contamination source of varying risk and hazard. Two indicators of hazard are the severity of the material being released and the size of the release. Guidance can only be given in a general form since specific circumstances control each case. The Resource Conservation and Recovery Act as a public law establishes mandates concerning the pollution of the environment and as such has direct relevance to this criteria. It sets a high seismic design level at which municipal waste facilities are to function to preclude contamination of the environment. This law requires that we place a high value on ground water and preclude contamination. As such this is probably the controlling relevant guidance for non-nuclear polluting or hazardous materials.

ESSENTIAL VS. ORDINARY CONSTRUCTION

Both Navy and civilian codes have made distinctions between ordinary and essential construction. *Generally, essential construction is expected to be operational after an earthquake. Facilities are deemed as essential by virtue of their need after an earthquake such as a hospital, fire station, or emergency recovery center. Navy facilities may be deemed essential by their mission requirement in support of national defense, such as a communication station. Utilities are usual deemed as essential based on the needs of fleet operability.* Piers and wharves are deemed as essential based on operational needs.

The decision to declare a structure as essential is to be made by the user in conjunction with the Naval Facility Engineering Command design agent. The decision to categorize a structure as essential results in a requirement for improved structural performance and a more restrictive damage state. The improved performance generally increases the initial construction costs perhaps by 10 percent for buildings but may reduce total life cycle costs considering both initial construction costs and expected damage costs over the life of the structure.

As part of this study, a survey was conducted of Navy fleet "managers" to solicit their opinions of design requirements. The types of people responding were ship's commanding officers, harbor management officers, operations officers, maintenance officers and staff civil engineers. A clear consensus of opinions indicates a desire not to have a Navy base be completely out of service for more than 2 weeks following an earthquake. They are willing to operate at 50 percent capacity for a period up to 1 or 2 months. Most respondents believe utilities are essential and must be operable within 2 weeks. The only way to be able to achieve these levels of operation, would be to categorize utilities and lifelines as essential and require enhanced performance. When asked to categorize a facility as ordinary or essential, all respondents rated hospitals and utilities as essential. A clear consensus rated piers, wharves, communications facilities, nuclear weapons facilities, and fuel farms as essential. Opinions for fleet ship and aircraft support maintenance facilities were divided. All respondents considered BOQs/BEQs and child care facilities as ordinary construction.

When considering a facility supporting an essential function, it is critical that the facility be considered as a system. It is not sufficient to consider a facility simply as a building structure, *but rather it is required to consider all the elements required to accomplish the mission to be accomplished in that structure. This usually includes requirements for electrical power, mechanical systems, water and sewer, communications, road access etc.* For example, a key communications section operated by a tenant command may have a requirement to send messages with no more than a 10 minute delay. To support that mission requirement, the operating agency utilizes a series of communication links which fall under the control of several other commands on a Navy base. The essential communication may initially utilize a telephone line, a modem switch bank, telephone substation, microwave transmission tower, a transmitter, and transmitting antenna. There are a number of links in the chain to accomplish the mission requirement of sending a message; each link must be evaluated. If the function is deemed essential, then each link in that chain becomes essential. Some lifeline chains are easy to identify; others require a careful search of routing charts, network diagrams and base maps. Since a Navy base is the size of a small city, it is critical that essential lifelines be clearly identified to prioritize repairs after an earthquake. This means that critical telephone circuits be marked or grouped to facilitate recovery. Most essential military mission functions are performed by tenant commands on a base; most repairs and recovery are or will be under the control of the base public works section. With a turnover of personnel, not all essential functions may be understood by public works support personnel. A team consisting of public works personnel and users should work together to develop a pre-earthquake plan for recovery and post-earthquake operation. It is essential that pre-earthquake planning identify all the key systems and develop prioritized recovery plans.

CURRENT NAVY CRITERIA

The Navy uses NAVFAC P355 for the design of buildings and associated details. Provisions are included in this reference for design of lifeline supports and equipment using 1990 SEAOC lateral force criteria. Chapter 11 presents a procedure for designing architectural elements. Chapter 12 addresses mechanical and electrical component anchorage while Chapter 13 deals with non-buildings and addresses tank design criteria. Chapter 14 gives an overview of utility systems and pipe details. The NAVFAC P355 (1992) does not reflect the most recent UBC and NEHRP provisions.

The general lateral base shear applied to a structure is the product of the structure weight, a zone coefficient, an importance coefficient, and a site factor composed of soil type and structure period, all divided by a ductility factor based on the type of lateral force resisting system. The pseudostatic load is distributed along the height of the structure and resulting stresses and overturning moments determined. Combinations of dead load, live load and other loads are used and orthogonal horizontal loads are combined to produce a total. Drifts are checked. Allowable stress design is used with adjustment factors.

The general, code anchorage force to be applied to a structure for relatively small elements of equipment within a building is specified as the product of the equipment weight, a zone factor, an importance factor, and a factor describing the type of element. The element is limited to less than 10 percent of the total weight of the structure or to 20 percent of the floor weight at the element level. Drift limitations can apply. For self-supported equipment on the ground, the value of F_P may be reduced by 2/3. Large elements are designed as non-buildings using the provisions of Chapter 13. Pipes containing hazardous materials within a building require special provisions for flexibility such as, flexible couplings, expansion joints, and spreaders.

The above is a brief overview of current criteria. Additional detailed information will be presented later in this report.

SEISMIC CODES RELATED TO LIFELINES

Current seismic codes when viewed as an ensemble, form a basis for understanding the state-of-the-art of risk quantification and the engineering profession's determination of prudent action. This section will present a number of seismic standards from which we may develop a consensus of design practice for Navy construction. DOE procedures and The Resource Conservation and Recovery Act (40 CFR 248 -USEPA 1991) both directly consider hazardous materials.[1]

1994 Uniform Building Code- The building code has been one of the origins for lifeline design under the category of non-building structures. The pseudostatic approach calculates an equivalent lateral force, V as;

$$V = [\,(ZIC) / R_w\,]^* \, W$$

$$C = 1.25\, S / T^{2/3}$$

where

T	Structural period < 2.75 seconds
Z	Zone factor
I	Importance factor = 1.0 to 1.25
S	Soil factor
R_w	Response modification factor or ductility factor

The Z factor represents the design earthquake ground acceleration according to the zone in which the structure is located and has a 10 percent probability of exceedance in 50 years. This is nominally a 500 year event or an event with an annual probability of exceedance of 2×10^{-3}. This design load is modified by the other factors of the equation; performance drift limits are used. The importance factor is used to increase the design load for important structures; however, the 1.25 is not large enough to produce elastic response under a severe earthquake. Wen et al (1994) notes that "this small range (in I) is hard to justify since the uncertainty in the seismic excitation is generally so large that the different reliability levels required of the structure would lead to a much larger range of the structural resistance. To determine the importance factor rationally and

[1] The following sections contain various code provisions. The nomenclature and notation of the original reference was kept the same and is not necessarily consistent among references.

qualitatively, a calibration of this value needs to be performed according to the performance goal required of the structure in terms of acceptable risks of limit states." The C factor is a function of the site soil conditions and the fundamental period of the structure.

The R_w factor allows for ductility in typical building structures and is also used for non-building elements. For non-buildings UBC Table 16-Q specifies R_w such as 3.0 for tanks. The ratio of C/R_w shall not be less than 0.5 The provisions call for computation of the lateral force of the tank using the entire weight of the tank and its contents. A response spectra approach allowing for inertial effects of the contents is permitted.

Lateral force on elements and components shall be designed for:

$$F_p = Z\, I\, C_p\, W_p$$

where C_p is specified in Table 16-O for elements such as tanks, racks, anchorage, plumbing etc. and W_p is the weight of the element. Rigid elements are designed for 0.5 of their weight times the Z and I factors.

For equipment in facilities drift must be checked. Drift limits are specified in terms of the interstory displacement divided by story height, d, as:

$$d = 0.03/R_w \text{ and } < 0.004$$

Wen et al (1994) notes the drift is about 1.5 percent independent of the response modification factor. This is not consistent with a reliability based approach.

1992 - 1994 NEHRP Provisions- The National Earthquake Hazard Reduction Program (NEHRP) has been used in waterfront design. The design earthquake is established as 10 percent chance of exceedance in 50 years which may result in both structural and non-structural damage which is expected to be repairable. For larger motions the intent is to preclude collapse. Peak ground acceleration maps are provided. The 1992 provisions computed seismic shear as:

$$V = C_s \; W$$

$$C_s = (1.2\, A_v\, S) / (R\, T^{2/3})$$

$$\text{but} \qquad C_s =< (2.5\, A_a)/R$$

where A_a and A_v are defined as effective peak acceleration and effective peak velocity-related acceleration. R is the response modification factor similar to the UBC but with different values. The 1994 provisions modified the equation by introducing amplification factors, F_a and F_v, and redefined the soil types into six groups:

$$C_s = (1.2\ A_v\ F_v) / (R\ T^{2/3})$$

$$\text{but} \qquad C_s =< (2.5\ A_a\ F_a)/R$$

The 1995 NEHRP soil site classes which establish values for F_a and F_v are defined as:

A) Hard rock with measured shear wave velocity, $v_s > 5{,}000$ ft/sec (1,500 m/s)

B) Rock with 2,500 ft/sec $< v_s <$ 5,000 ft/sec (760 m/s $< v_s <$ 1500 m/s)

C) Very dense soil and soft rock with 1,200 ft/sec $< v_s <$ 2,500 ft/sec 360 m/s $< v_s$ 760 m/s) or with either $N > 50$ or $s_u \geq$ 2,000 psf (100 kPa) where N is average blow count SPT and s_u is average undrained shear strength

D) Stiff soil with 600 ft/sec $< v_s <$ 1,200 ft/sec (180 m/S $< v_s <$ 360 m/s) or with either $15 < N < 50$ or 1,000 psf $< s_u <$ 2,000 psf (50 kPa $< s_u <$ 100 kPa)

E) A soil profile with $v_s <$ 600 ft/sec (180 m/s) or any profile with more than 10 ft (3 m) of soft clay defined as soil with $PI > 20$, $w > 40$ percent, and $S_u <$500 psf (*25* kPa)

F) Soils requiring site-specific evaluations:

1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils.

2. Peats and/or highly organic clays (H > 10 ft (3 m) of peat and/or highly organic clay where H = thickness of soil)

3. Very high plasticity clays (H > 25 ft (8 m) with PI > 75)

4. Very thick soft/medium stiff clays, H> 120 ft (36 m)

EXCEPTION: When the soil properties are not known in sufficient detail to determine the Soil Profile Type, Type D shall be used. Soil Profile Types E or F need not be assumed unless the regulatory agency determines that Types E or F may be present at the site or in the event that Types E or F are established by geotechnical data.

The 1995 NEHRP provides the following steps for classifying a site.

Step 1: Check for the four categories of Soil Profile Type F requiring site-specific evaluation. If the site corresponds to any of these categories, classify the site as Soil Profile Type F and conduct a site-specific evaluation.

Step 2: Check for the existence of a total thickness of soft clay > 10 ft (3 m) where a soft clay layer is defined by: s_u < 500 psf (25 kPa), w > 40 percent, and PI > 20. If these criteria are satisfied, classify the site as Soil Profile Type E.

Step 3: Categorize the site using one of the following three methods with v_s, N, and s_u

a. v_s for the top 100 ft (30 m)

b. N for the top 100 ft (30 m)

c. N_{ch} for cohesionless soil layers (PI < 20) in the top 100 ft (30 m) and average s_u for cohesive soil layers (PI > 20) in the top 100 ft (30 m) where N_{ch} is average blowcount for cohesionless layers from SPT

The NEHRP provisions found in FEMA 223A Section 3.3.9 discusses storage tanks and allows either the AWWA or the API procedures. It specifies that pipe connections to steel storage tanks provide for 2 inches of vertical displacement for anchored tanks and 12 inches for unanchored tanks. It further specifies piping systems to be made of ductile materials; design strengths for service load combinations may be 90 percent of yield strength for ductile steel, aluminum, or copper, 70 percent of yield strength for threaded pipe made from ductile material and 25 percent of minimum tensile strength for plastic pipe. Threaded connections in piping constructed of nonductile materials shall not more than 20 of minimum specified tensile strength. Section 3.1.3 defines seismic force levels for tanks and piping.

DOE Criteria -The Department of Energy (DOE) guidelines define four classes of structure, General Use, Important / Low Hazard, Moderate Hazard, and High Hazard. The later two classes refer to nuclear facilities. This work establishes a risk acceptability criteria which has direct correlation to the containment of hazardous materials and lifelines. General Use facilities are typical ordinary structures to be designed by current code provisions. Important / Low Hazard facilities would include laboratories, computer centers, hazard recovery facilities and other facilities with a building code importance factor of 1.25. Moderate Hazard facilities include facilities where confinement of contents is necessary to protect personnel including the handling of radioactive and toxic materials. High Hazard facilities include facilities where confinement of contents is necessary for public and environmental protection such as nuclear facilities; these facilities represent hazards with potential long term and widespread effects. Specification of the design earthquake is established in terms of the annual probability of exceedance starting at a value similar to the UBC value and decreasing with class of structure. The annual probabilities of exceedance values expressed as earthquake nominal return times used for the four classes of structure are:

Structure Class	Earthquake Return Time (years)	Annual Exceedance Probability
General Use	500	2×10^{-3}
Low Hazard	1000	1×10^{-3}

Moderate Hazard	1000	1×10^{-3}
High Hazard	5000	2×10^{-4}

The DOE guidelines define the performance goals of each class of structure. Associated with each performance goal is a probability of the structural system meeting that goal. The following table shows that relationship.

Structure Class	Performance Goal	Probability of failure to meet goal
General Use	Occupant Safety Prevent major structural damage/collapse Code provisions	0.5
Low Hazard	Continued Operation Capacity to function, occupant safety, relatively minor structural damage	0.5
Moderate Hazard	Continued Functionality Hazard Confinement Limited damage to insure containment of hazardous materials	0.1
High Hazard	Continued Functionality Hazard Confinement Limited damage to insure containment of hazardous materials	0.05

By combining the earthquake probability of occurrence and the probability of exceedance of failure shown above the annual probability of failure can be calculated and is shown below. These values range from 0.001 for general use structures in which the measure of performance is probability of collapse to 0.00001 for high hazard facilities in which the measure of performance is failure of containment of the high hazard. The goals and probabilities are:

Structure Class	Performance Goal	Annual Exceedance Probability of Failure
General Use	Occupant Safety	1×10^{-3}
Low Hazard	Continued Operation	5×10^{-4}
Moderate Hazard	Continued Functionality Hazard Confinement	1×10^{-4}
High Hazard	Continued Functionality Hazard Confinement	1×10^{-5}

This data is thought to be a significant statement on the general acceptability of seismic risk and as such has direct bearing on establishing guidance for comparable operations associated with Navy facilities.

40 CFR 248 -USEPA 1991 The Resource Conservation and Recovery Act - This public law specifies the design requirements for municipal solid waste landfills (MSWLF). There is major concern that these dumps can pollute the ground water. The law states "new MSWLF units and lateral expansions shall not be located in seismic impact zones unless... all containment structures, including liners, lechate collection systems and surface control systems are designed to resist the maximum horizontal acceleration in lithified earth material for the site" The law mandates that a composite liner composed of a geomembrane and 2 feet of low permeability soil be used. The maximum acceleration is defined as emanating from a seismic event with a 90 percent chance of not being exceeded in 250 years; this is nominally a 2500 year return time event. Design criteria is given for allowable concentrations of toxic chemicals and acceptable values of hydraulic conductivity.

This legislation is a significant statement which establishes defined risk limits for seismic pollution of the environment and as such is applicable to comparable Navy facilities.

American Water Works Association D100, D103, D110 Standards. These standards describe the design of bolted and welded steel tanks and prestressed concrete tanks. Structures to be designed for Seismic Zones 1, 2 or 3 may be designed for a fixed percentage weight of 2.5 percent, 5 percent and 10 percent respectively. Elevated tanks are design using:

$$V = Z\,KC\,SW$$

where

$$C = 1/(15\sqrt{T})$$

Specific values for K are given.

Tanks on the ground in Seismic Zone 4 require a pseudostatic design but allow for a response spectra. The horizontal base shear is given by:

$$V = Z\,I\,K\,S\{C_1\,(W_s + W_r + W_l\,) + C_2\,W_2\,\}$$

and the overturning moment is given by:

$$M = Z\,I\,K\,S\{C_1\,(W_sX_s + W_rH_t + W_lX_l) + C_2\,W_2X_2\,\}$$

Where

C_1 Factor based on natural period
C_2 Factor based on natural period

H_t Total height of tank shell
I Importance factor
K Structure coefficient depends on type and anchorage
S Soil factor
W_r Weight of effective mass of tank roof
W_s Weight of effective mass of tank wall
W_1 Weight of effective mass of tank contents moving with tank shell
W_2 Weight of effective mass of first mode tank sloshing
X_s Height from bottom of tank shell to cg of shell
X_1 Height from bottom of tank shell to centroid of lateral force applied to W_1
X_2 Height from bottom of tank shell to centroid of lateral force applied to W_2
Z Zone factor

The bolted steel tank standard uses an SC_1 value of 0.14. The fundamental period of the tank is prescribed by equation and varies depending on the particular standard and tank type. A fundamental period for the sloshing mode is also computed. The K value varies with the type of tank and whether it is anchored or not. Unanchored tanks have higher K values.

Response spectra values can be substituted for equation values. The approach considers that the loading consists of components at the tank fundamental frequency and also components at the sloshing frequency. Response spectra values based on a tank period can be substituted for $ZIKSC_1$. Additionally, sloshing period values can be substituted for $ZIKSC_2$. Vertical force components can be included in the computation. The designer has the option to compute the resultant separately or in conjunction with horizontal forces. Tank wall stresses are computed from overturning moments and compared with allowable values. Formulas are given for computation of vertical compressive and tensile forces at the tank base. Flat-bottom tanks may be anchored or unanchored. Where tanks are unanchored the maximum thickened annular ring width at the base used to limit overturning is limited to 7 percent of the tank radius and the thickness shall not exceed the thickness of the shell thickness at the bottom. Anchored tanks could be susceptible to tearing if not properly designed. Hydrodynamic seismic tensile membrane forces are computed. Allowable stresses are increased by one-third for seismic forces. Guidelines are given for important foundation considerations including allowable bearing and the need for soil homogeneity across the foundation. Various types of tank foundations are discussed. The user shall specify the amount of tank freeboard for sloshing. Failure to provide for sloshing will damage the roof if the tank is completely full. Provisions are included to allow for local site conditions. A 2 percent damped curve is recommended for design of the structure and a 0.5 percent damped curve is recommended for sloshing of the liquid. The amplified acceleration shall be determined for the cantilever beam period of the shell and effective portion of the contained fluid. When site response return times are not given a maximum credible event or 10,000 year return time event can be used with a response reduction factor not to exceed 2.6.

The AWWA has standards for ductile iron, steel, concrete, and asbestos pipe; however they do not address seismic design directly.

American Petroleum Standard 650- The American Petroleum Institute provisions follow 1980's code design and was revised and updated as recently as 1996. The tank overturning moment is:

$$M = Z\,I\,\{C_1\,(W_s X_s + W_r H_t + W_1 X_1) \;+ C_2\,W_2 X_2\,\}$$

where the terms are the same as defined for the AWWA equation above. The term C_1 is set at 0.60 unless the product of $Z\,I\,C_1$ and $Z\,I\,C_2$ are determined from response spectra. The term C_2 is defined by:

$$C_2 = 0.75\ S\,/\,T \qquad \text{for } T <= 4.5$$

$$C_2 = 3.375\ S\,/\,T^2 \qquad \text{for } T > 4.5$$

If a spectrum is used for the factor $Z\,I\,C_1$, it should be developed for a damping coefficient of 2 percent of critical. The spectrum for the factor $Z\,I\,C_2$ should be based on the spectrum for $Z\,I\,C_1$ but with a damping coefficient of 0.5 percent of critical.

Summers (1997) reports that extensive experimental studies and observations during past earthquakes have demonstrated that the radial length of uplifted bottom plate, and hence, the actual liquid weight resistance which is mobilized during an earthquake is underestimated by the API uplift model. He explains the reasons for this are that the API model does not account either for the in-plane stress in the bottom plate, or for the dynamic nature of the tank response. The API model also calculates a somewhat narrow compression zone at the toe of the tank, thus leading to large compressive stresses in the tank shell for relatively low overturning moments. Finally, the API approach does not account for the effect of foundation flexibility on the tank wall axial membrane stress distribution. These factors err on the conservative side and result in overdesign. The API procedure is recognized as a conservative approach and is acceptable for new tank design.

40 CFR 112: 38FR 34164 Environmental Protection Agency Regulations On Oil Pollution Prevention This public law applies to oil storage or processing facilities which are potential pollution sources. It does not apply to facilities where the storage capacity is 1,320 gallons or less and no single tank has a capacity in excess of 660 gallons. For facilities falling under provisions of this law, appropriate secondary containment is mandated such as dikes, curbs, sumps or ponds.

State of California Above Ground Storage Act of 1991 This law applies to sites containing petroleum/hazardous material storage tanks where the above ground storage capacity is over 1,320 gallons or where a single tank exceeds a capacity of 660 gallons. The law requires inspections, licensing and monitoring. The foundation system must be designed to allow for early detection of releases of materials before reaching the ground water.

American Railway Engineering Association, Chapter 9 Seismic Design For Railway Structures The procedure specifies three levels of ground motion: A Level 1 ground motion has a reasonable probability of being exceeded during the life of the structure and the structure is at a serviceable limit state which requires it to remain elastic. Only moderate damage which does not affect trains at restricted speeds is allowed. Allowable stresses are increased 150 percent in steel and 133 percent in concrete elements. The return period for a Level 1 earthquake is between 50 and 100 years. The determination of a specific ground motion level is left to the designer based on the type and volume of traffic expected. A Level 2 ground motion has a low probability of being exceeded during the life of the structure and represents a limit state to ensure overall structural integrity. The structure may respond in the inelastic range but ductilities are limited. The return period for a Level 2 earthquake is between 200 and 500 years. The selection of the specific level is left to the designer based on overall economics considering structure cost and train schedules. A Level 3 ground motion is established for a rare intense earthquake which establishes a survivability limit state which allows extensive damage but precludes collapse. Foundation failures are limited so as not to cause major changes in the structure geometry. The return period for a Level 3 earthquake is between 1000 and 2500 years. The selection of a specific level is left to the designer based the consequences of loss of the structure and include costs of construction, loss of use, existence of alternate routes and location of the bridge. Pseudostatic, spectral and dynamic procedures are used depending on the type and irregularity of the structure. The nominal 100 year, 500 year and 2500 year return time peak horizontal rock accelerations are specified on a national map

Standard Specification For Highway Bridges, AASHTO This is a national code and as such divides the US into regions based on levels of expected ground motion. A map is provided which shows peak horizontal rock accelerations with a 90 percent probability of not being exceeded in 50 years which is a nominal 500 year return time event. Two categories of bridge structure are defined, essential bridges which are expected to function after a design earthquake and other bridges which are designed for near elastic response at moderate events and for limited damage at the maximum credible event. Four categories A through D are defined to treat importance and variation in seismic acceleration potential. A and B are low treat level requirements while D is highest representing an essential structure in the highest exposure zone. Three site profiles are defined and serve to define site amplification. Elastic earthquake lateral forces are determined based on the map accelerations and site soil factor. Component response modification factors are used to reduce the elastic forces for substructure elements while connections of superstructure to abutment and expansion joints are increased. The modification factors are analogous to ductility factors. It is assumed that columns will yield when subjected to forces from the design ground motion but that the connection will be able to resist the deformations with little damage. Wall piers have minimal ductility and an R value of 2 was assigned. Well designed columns in a multi-column bent have good ductility and a value of 5 was assigned to them. Single columns lack redundancy thus a value of 3 was assigned. For C and D bridges the connections are designed for the maximum forces that can be developed by plastic hinging in the columns. The probability of elastic force levels not being exceeded in 50 years is in the range of 80 to 90 percent. Procedures are given to calculate displacements. Modal response techniques are used in the analysis of response. It is suggested that a factor of safety against liquefaction be 1.5 for important bridges. Guidance is given for pile design

1990 CALTRANS CALTRANS criteria was developed for non-buildings and is of general interest. It is summarized as:

$$V = ARS\ \ W/Z$$

where W is the total weight and Z is an adjustment factor for ductility and risk and based on the period and type of structural element. ARS is the 5 percent elastic response spectrum at the site in g's based on the maximum expected acceleration at bedrock or rocklike material. The seismic force in two directions is required and to be evaluated by adding 30 percent of the force to the component in the perpendicular direction. For conservative design, the vector sum can be used. A load factor of 1.0 is used and live load is not included. The strength reduction factor, ϕ, for concrete columns can be increase from 0.9 to 1.2 to recognize an increase in strength from well confined concrete.

Japan Gas Association Recommended Practice For Pipelines- The 1978 Miyagiken-oki earthquake caused heavy damage to the gas distribution system in Sendai City. Damage was concentrated in threaded steel pipelines of about 2-inch diameter. As a result of this guidelines were developed for Japan. Japan is divided into four seismic zones and three soil classifications are used. The seismically induced horizontal ground deformation is estimated by:

$$U = \alpha_1 \alpha_2 \ U_0$$

where

α_1 Constant based on site location in the range of 0.4 to 1.0
α_2 Constant based on soil condition and importance in the range of 0.5 to 1.8
U_0 Constant which is set at 5 centimeters

The vertical displacement is half of the horizontal. The guide outlines four load deformation conditions shown in Figure 1 and a Deformability Index is used to estimate pipe capacity. The Deformability Index includes strain capacity of the pipe and of the joint.

IEEE Standard 344-1987- The IEEE has developed a standard for the seismic qualification of equipment for the nuclear industry.

PERFORMANCE OBJECTIVES

The development of performance objectives is the first step in development of a general criteria for lifelines. *The following performance objectives are presented herein and represent a new synthesis proposed for Navy use. They are based on mandates of public law and extensions of current Navy criteria.*

Ordinary Construction / Ordinary Lifelines - *Lifeline service associated with construction categorized as "ordinary" shall be designed with the same levels of service. In general ordinary construction is expected to*

- *Resist a minor level of ground motion without damage;*

- *Resist a moderate level of ground motion without structural damage, but possibly experience some nonstructural damage;*
- *Resist a major level of earthquake (10 percent probability of exceedance in 50 years) ground motion without collapse, but with structural as well as nonstructural damage.*

Essential Construction / Essential Lifelines *- Lifeline service associated with construction categorized as "essential" shall be designed with the same levels of service. In general essential construction is expected to:*

- *Resist the maximum probable earthquake likely to occur one or more times during the life of the structure (50 percent probability of exceedance in 50 years) with minor damage without loss of function and the structural system to remain essentially linear.*
- *Resist the maximum theoretical earthquake with a low probability of being exceeded during the life of the structure (10 percent probability of exceedance in 100 years) without catastrophic failure and a repairable level of damage.*

Hazardous Materials/Lifelines *- Lifeline service associated with construction categorized as "containing hazardous materials" shall be designed with the same levels of service. In general hazardous material containment construction is expected to:*

- *Conform with criteria for essential construction*
- *Resist pollution and release of hazardous materials for an extreme event (10 percent probability of exceedance in 250 years)*

SEISMIC LOADS

The second element of a general criteria for lifelines is the specification of seismic load level to establish the ground motion and lateral load forces to be applied in design. *It is based on current criteria and an extension of existing mandates logically applied to analogous situations.*

Design Earthquakes

The following criteria are based on current Navy criteria and public law. The Navy lifeline systems shall be designed to resist the loading produced as follows:

- ***Ordinary category of construction on average seismicity sites***
  For sites of average seismicity, use NEHRP, UBC and NAVFAC P355 provisions, which establishes the earthquake at a nominal 10 percent chance of exceedance in 50 years.

- ***High seismicity or essential category of construction***
  Sites of high seismicity controlled by local faulting where general code provisions do not account for local hazard potential, or where the lifeline is deemed important and essential shall use a two-earthquake procedure with Level 1 earthquake having a 50 percent probability of exceedance in 50 years and a Level 2 earthquake having a 10 percent

probability of exceedance in 100 years based on a local site seismicity study. Values less than code are not be permitted.

- ***Construction containing polluting or hazardous material***
  A Level 3 earthquake having a 10 percent probability of exceedance in 250 years exposure shall be used.

As part of this criteria:

- *the determination of the design earthquake shall be performed using techniques described in criteria for ground motion for essential structures*

In addition to seismic ground motion there are additional hazards which must be considered:

- *Fault movement and ground displacement*
- *Liquefaction and associated lateral spreading, settlement flow slides, loss of support and buoyancy of buried tanks.*
- *Landslides*
- *Tsunamis*

Modification to Design Ground Motion

Lifelines consist of a variety of elements some of which are substantial structures such as tanks, transformer stations and bridges, others are distributed elements such as buried pipelines, power lines and railroad tracks, and others are components within structures such as internal equipment, transformers, and other building elements. *The ground motions used in design of lifelines may differ from the motions used in conventional building design since the seismic motion on the lifeline may be substantially different than that associated with free-field ground motion. For component elements located within a structure the lifeline component design motion can be substantially amplified by the response of the structure. In such cases the motion to be used for design of the component must be the local seismic motion transmitted by the structure to the component.* The dynamic coupling between the component and the structure must be taken into account if the component is of a size sufficient to influence the response of the structure. Large differential motions may be produced on components which are supported at multiple locations.

Chapter 6 of NAVFAC P355.1 illustrates the procedure for calculation of the maximum floor accelerations using the linear response spectra technique. A modal participation factor is applied to the story modal acceleration response to determine the modal spectral acceleration to be applied to the lifeline component. A design response spectrum is constructed using the modal floor accelerations, the participation factors, a magnification factor, and the period of the lifeline component. Response spectra techniques have been utilized for at least the last 35 years. They offered a means for performing dynamic analysis more accurately than pseudostatic approaches. The response spectra technique is a linear procedure. A structure responding to a Level 2 earthquake is expected to sustain significant nonlinear behavior. The ability of response spectra techniques to accurately track displacements reduces as the amount of nonlinearity increases. With the evolution of the desktop computer, nonlinear finite element techniques which previously required extensive mainframe computer time, have now been developed which can offer a potentially more accurate analytical alternative.

LIFELINE PERFORMANCE DURING RECENT EARTHQUAKES

Understanding the behavior and possible failure mechanisms of a lifeline structure is important in the development of a design criteria for safe operation. Part of understanding the performance of a lifeline structure in an earthquakes involves understanding the design from which the structure was constructed. and the construction practice used in its erection. Werner and Hung (1982) gives an excellent compilation of case studies mostly recounting Japanese experiences from the 1920's to 1980. They conclude that "By far the most significant source of earthquake-induced damage to port and harbor facilities has been porewater pressure buildup... which has led to excessive lateral pressures applied to quay walls and bulkheads." They cite the 1964 Niiagata and 1964 Alaska earthquake where "porewater pressures buildup has resulted in complete destruction of entire port and harbor areas" They note that direct effects of earthquake induced vibrations on waterfront structures is minimal and overshadowed by liquefaction induced damage. In the 1978 Sendai earthquake a major oil refinery with 90 storage tanks had three fail and three damaged. Additionally a large welded steel plate tank pulled out of its concrete embedment. A summary of recent lifeline experiences during earthquakes follows.

Alaska Earthquake - The 1964 caused considerable damage to oil storage tanks by tsunamis, earth settlement, and liquefaction. Damage to Union Oil tanks in Whittier caused fires. In Anchorage seven tanks collapsed releasing combustible fluids; three additional Standard Oil tanks released 750,000 gallons of aviation fuel. This experience led to a change in tank design, Eguchi (1987)

San Fernando Earthquake- The 1971 San Fernando earthquake resulted in direct losses to the electric power systems of $33 million. It caused distress to numerous tanks. Bulging of the lower section of about 12-inches above the base was noted extensively and termed "Elephant's Footing". Ductile steel pipelines were able to withstand ground shaking but could not withstand ground deformation associated with faulting and lateral spread. Eleven transmission pipelines were damaged by liquefaction induced lateral spread and landslides. Eighty breaks occurred to the underground welded steel transmission pipeline located in the upper San Fernando Valley, the most serious in a 1930 old oxyacetylene-welded pipeline. Although located in an uplift zone the failure was caused by compressive forces wrinkling the pipe, Eguchi (1987). Newer pipelines in the same area did not fail. There were 18 documented hazardous material releases resulting in 6 fires. There was damage to the Jensen water treatment plant resulting in an outage. The absence of an inlet to outlet bypass was noted as a factor in impeding the problem of restoration of service.

Santa Barbara - In the 1978 magnitude 5.1 Santa Barbara earthquake a train derailed shortly after the earthquake from damaged tracks. About 40 cars derailed at a speed of 50 miles per hour.

Coalinga Earthquake - The 1983 Coalinga earthquake had adequately designed pipelines which remained serviceable; however large vertical tanks containing molasses tilted and in one case overturned. This was initiated by large deformations in the steel support frame. At a treatment plant, chlorine tanks on standard saddle supports slid up to 10 inches. The valves on a 1-inch line to a clarifying tank shook open causing a major oil spill. Anchors on a 12 kV transformer broke. A hazardous material spill resulted in significant damage to a high school; there were three other hazardous material incidents of significance. Numerous breaks in the natural gas line occurred but fires did not occur since the main valve was closed manually shortly after the earthquake. Several tank and pipeline failures occurred in oil drilling and processing facilities. In general it was noted that secondary containment systems functioned well. most pipe breaks occurred at pipe connections.

Whittier Narrows- The 1987 magnitude 6.1 earthquake demonstrated that well designed process pipelines can perform well. Damage where it occurred was usually limited to sections that were corroded or anchored at two locations which experienced large lateral relative displacement. A 1-ton relocatable gas cylinder being filled with chlorine started to roll down the loading platform breaking the connection causing a significant chlorine release. Southern California Gas reported 1411 gas leaks were directly caused by the earthquake. Portions of the California State University, Los Angeles were without gas for 12 weeks. Five fires were reported; three of these were attributed to gas leaks. There were about 30 hazmat calls for assistance.

Loma Prieta Earthquake- The 1989 magnitude 7.1 Loma Prieta Earthquake caused failure of many pipelines and tanks. The Port of Redwood City is located at the southern end of the San Francisco Bay. The Port contains tanks for petroleum. The Port was constructed on Bay Mud. Damage consisted primarily of broken water lines and damaged batter piles. The Port of Richmond is located at the northeast end of the San Francisco Bay and handles petroleum products and liquid bulk cargo. Portions of this port are constructed on rock and other portions on fill. The primary damage was the rupture of a gasoline storage tank at the UNOCAL terminal. Fuel was contained in the surrounding berm. Some liquefaction was reported in undeveloped areas of the port. Broken waterlines occurred at the Ford plant from liquefaction and excessive soil pressures. The Port of San Francisco is located on the west side of the San Francisco Bay and handles general cargo. The port is constructed on fill. The primary damage was liquefaction and settlement. Numerous buildings were damaged water and gas lines broke. The Port of Oakland is located on the east side of the San Francisco Bay on fill. The port sustained wharf damage and noted batter pile failures. Liquefaction of the fill produced settlement and lateral spread. Horizontal accelerations were measured at the wharf and ranged from about 0.3g to 0.45g. Cranes suffered damage and water lines broke. Fire lines ruptured eliminating fire fighting protection, Seed et al. (1990). Tank failure modes consisted of "elephant's foot " bulging, vertical splitting of tank wall, puncture of the tank wall by restrained pipe, pipe damage from differential anchorage motion. Hazardous material spills occurred in several industrial and a few commercial facilities. Over 300 liquid hazardous spills occurred in the San Francisco and Monterey Bay areas as a result of ruptured tanks, pipe leaks, equipment leaks, and broken containers. It appears that secondary containment was generally effective. At least 50 instances occurred of release of hazardous gases other than natural gas. There were 3 to 4 leaks on a high pressure gas main and between 300 to 400 leaks on low pressure gas lines.

The Navy sustained 44 pipeline breaks in pipes up to 16 inches in diameter on Treasure Island. They included 28 fire and freshwater lines of steel or asbestos cement, 10 sewage lines of vitrified clay and 6 welded-steel gas lines, Egan and Wang (1991). Many of the breaks occurred near the dike in areas of high lateral spreading. Crude estimates of lateral spreading required to cause failure are:

Type	Pipe Diameter	Spreading to Induce Failure
Steel or Asbestos Cement	1 to 4 in	1 inch
Steel or Asbestos Cement	12 to 16 in	6 to 12 inches
Vitrified clay pipe		1/4 inch

Soil liquefaction caused damage to the terminal facilities much of which were on filled land composed of loose dumped or hydraulically placed sand underlain by soft normally consolidated Bay Mud. Liquefaction of the fill resulted in settlements and lateral spreading, cracking the pavement over a wide area. Maximum settlements of the paved yard area were up to a 12 inches.

In the Monterey area water tanks belonging to PG&E were damaged and one ruptured apparently as a result of foundation softening and displacements. Settlements of several inches were noted and there were breaks in utility lines. Pile supported facilities were not damaged.

Transportation facilities sustained $1 billion in damage including $200 million to the Cypress Street elevated viaduct. Numerous roads were closed by pavement damage, landslides, or bridge damage. A 3000-foot section of runway was severely damaged having several breaks as large as 30 inches in width. Undulations were noted in the pavement along with settlement. The pavement was situated was on 10 to 15 feet of unconsolidated hydraulically dredged sand fill which experienced extensive liquefaction. The runway at the Naval Station, Alameda cracked and moved laterally from liquefaction of the soil below. The Port of Oakland experienced liquefaction damage to paved yard areas Batter piles in wharves were damaged.

Big Bear Earthquakes- On June 28, 1992 two earthquakes occurred in San Bernadino County, California, a magnitude 7.5 at 4:58 AM and a magnitude 6.6 at 8:04 AM. These two events were followed by numerous aftershocks. Horizontal fault rupture displacement associated with these event was from 5 to 9.5 feet. Most pipeline damage was associated with the rupture zone. At least 6 water tanks ranging in size from 42,000 to 417,000 gallons were damaged. Damage consisted of elephant's foot bulging at the base, shell and roof damage, shell splitting at access hatches and broken pipe entering the tanks.

Guam Earthquake - On August 8, 1993 a magnitude 8.1 earthquake occurred 50 miles offshore and caused over $125 million in damages to Naval facilities on Guam. Nearly all of Guam is firm soil or rock except for the region containing the commercial and Navy ports which is composed of natural alluvium and artificial fill. It is estimated the peak horizontal ground accelerations were about 0.25g. Liquefaction was a major problem and lateral spreading of 1 to 2 feet was observed at wharf areas. It also resulted in settlements, backfill collapse and bulkhead movements. Buried water and power lines were fractured. Sheet piles failed in shear and deadman anchors pulled out. Pier batter piles failed in shear at the pile cap. Other Navy damage consisted of fuel tank leaks, sloughing of a dam, damage to masonry housing units and major damage to the power plant which supplied 20 percent of the islands power capacity.

Northridge Earthquake- On January 17 1994 a moment magnitude 6.7 earthquake occurred in Northridge. This event caused about 1,400 water, gas and fuel pipeline breaks in the San Fernando Valley area. Many of the breaks occurred in mapped areas of high liquefaction potential. Outside the zone of high liquefaction potential, the dispersed pattern of breaks is attributed to old brittle pipes damaged by ground movement. While much of the pipe damage is within the liquefaction zone, this did not correlate to areas of high structural damage in that a large amount of structural damage occurred outside the zone of high liquefaction potential. In the Granada Hills area pipe breaks from water mains resulted in soil erosion and formation of large craters. On Balboa Boulevard a 22- inch pipe suffered two breaks, one in tensile failure and the other in compressive failure. These pipe failures were located in a ground rupture zone perpendicular to the pipeline. Leaking gas ignited at several locations. Some broken water and gas lines were found to have experienced 6 to 12 inches of separation in extension. The area experienced widespread ground cracking and differential settlements. Liquefaction was not evident on the surface and may have occurred at depth leading to subsurface soil block movement. Some of the surface cracking was associated with underlying bedrock movements associated with primary or secondary faulting. A 85 inch sewage pipe ruptured in the Jensen Filtration Plant and a large reservoir settled 2 to 4 inches. The San Fernando Power Plant Tailrace, a 600 by 110 foot asphalt lined pond was breached. Lateral spreading was noted. A water storage tank east of Highway 5 at Valencia Boulevard collapsed. The Port of Los Angeles sustained peak horizontal accelerations on the order of 0.1 to 0.2g which resulted in liquefaction of hydraulic fill damaging crane rails, disruption of utilities, ground cracking and lateral spreading of up to 6 inches. All of the damage was of a relatively minor nature.

Kobe Earthquake- On January 17 1995, the Hyogo-ken Nambu (Great Hanshin Kobe) earthquake, Japanese magnitude 7.2 (about 6.9 moment magnitude), occurred in Kobe Japan. This event produced major damage to Japan's second busiest port, Matso (1995). Liquefaction was a major contributor to the extent of the damage producing typical subsidence of a half meter. Piles were used extensively in this area. They were designed to account for the negative skin friction and additional ground improvement was also performed. Structures on such piles performed well even though major subsidence occurred in surrounding areas. Other structures not on piles suffered differential settlement and tilting and significant damage. Liquefaction caused up to 3 meters of lateral spread displacement, sunk quay walls, broke utility lines, and shut down 179 out of 186 berths at the port. Numerous tank failures were reported, mostly caused by uplift of unanchored tanks. One LNG tank cracked requiring the evacuation of 80,000 people. Six well-braced large spherical tanks sustained no damage. Liquefaction was responsible for major damage to crane foundations. Hydraulic fill behind concrete caisson perimeter walls fill liquefied causing the caissons to move outward, rotating up to 3 degrees, and settling from 0.7 to 3.0 meters. The caissons were designed for a lateral coefficient of 0.1g. A seismic coefficient of 0.2g was usually used in the design of dockside cranes. Peak accelerations of 0.8g in the NS direction, 0.6g in the EW direction and 0.3g vertical were noted from accelerograph recordings. The event had a duration of about 20 seconds. Most damage is attributed to liquefaction of backfill and associated pressures and settlements and lateral deformations since structures supported on piles suffered much less damage, Liftech (1995). It should be noted that caissons designed for 0.25g sustained lower levels of damage.

LIQUEFACTION AND LIFELINES

Design of structures shall include provisions to evaluate and resist liquefaction of the foundation and account for expected potential settlements and lateral spread deformation.

As part of this criteria:

- Retaining structures shall be designed using provisions in NCEL Technical Report R-939.
- A separate Navy criteria document on liquefaction.

Special care will be given to buried pipelines in areas subject to liquefaction to preclude breaks resulting in release of hazardous materials. The most important element in seismic design of pipelines is proper siting. It is imperative to avoid areas of landslide and lateral spread.

The presence of any potentially liquefiable materials in foundation or backfill areas shall be fully analyzed and expected settlements computed. Specific attention shall be paid to the acceptability of the amount of settlements. Since liquefaction is a major damage mechanism at the waterfront, remediation is a mandatory requirement where the risk of a pipeline break or tank failure is shown by computation to be possible and hazardous materials would be expected to be released.

WATER, GAS AND LIQUID FUEL LIFELINES

Pipelines

Pipelines must be designed to resist the expected earthquake induced deformation state and induced stresses. It is common practice to design the pipe support or embedment based on the nature of the soil encountered. Where marginal soil is encountered, pipelines can be supported on piles above ground or placed within larger pipes to allow ground movement. *Generally permissible tensile strains are on the order of 1 to 2 percent for modern steel pipe.* This is based on observation that steel pipelines have been observed not to rupture at tensile strains between 2 to 5 percent. Higher local strains have been noted. Pipelines have experienced wrinkling in compression at strains much less than the tensile limits; however this does not of itself constitute failure. A rule of thumb states that the onset of wrinkling occurs at strains of about 0.3 times the ratio of wall thickness to radius. Welded steel pipes have performed well during earthquakes. The quality of weld is very important. There appears to be more failures with oxyacetylene-welded steel pipes compared to arc-welded steel pipes. The difference may not be the type of weld but may be the weld quality. Corrosion of pipelines reduces their ability to withstand seismic forces. Pipeline damage seems inversely proportional to pipe diameter caused by an increase in stiffness with larger size pipe which makes it more able to resist deformation. Expressed in another form, pipeline strength is proportional to diameter. An exception to this seems to be steel pipe with a lap welded joint where strength decreased with increasing diameter. Also gasketed joints seem to be 5 or more times more likely to fail than welded joints. In addition to tensile and compressive failures, buckling failures are possible. The presence of bends, elbows and local eccentricities tends to concentrate deformation at these locations.

To accommodate differential motion between pipelines and storage tanks it is recommended that a length of pipeline greater than 15 pipe diameters extend radially from the tank before allowing bends and anchorage and that subsequent segments be of length not less than 15 diameters. Flexible couplings should be used on long pipelines. In general pipes should not be fastened to differentially moving components; rather, a pipe should move with the support structure without additional stress. Unbraced systems are subject to unpredictable sway whose amplitude is based on the system fundamental frequency, damping and amplitude of excitation. For piping internal to a structure, bracing should be used for system components. Flexible grooved pipe couplings can reduce the transmission of stresses and resilient gaskets can dampen vibration. Manufacturers specification give guidance on linear and angular movement tolerances. "Grooved-end mechanical pipe couplings do not simultaneously provide maximum linear and angular movement. However, systems designed with enough joints, thus allowing for recommended tolerances, will accommodate both", Greene (1993). When large movements are anticipated seismic swing joints composed of flexible couplings, elbows and nipples can be used. Provisions for expansion must be included.

Machinery and pumps are often acoustically isolated by use of loose connections to minimize vibration transmission such as by use of slotted holes. *Snubbers by definition are restraints with an air gap. Such anchorages can amplify seismic motion by having equipment bang against restraints. Use of resilient grommets or molded epoxy grouting can eliminate the air gap and avoid hard surface contact. The snubber and the connection of the snubber to the equipment and structure must have sufficient strength to transmit the inertial forces.* The Northridge earthquake has shown that use of rails is not a satisfactory method of restraint and such usage was associated with many failures of welds and dislocation from the rails. Suspended pipelines can also resonate with the earthquake if not sufficiently restrained. Sway of suspended components must be restrained. *Seismic isolation can be an effective technique for reducing loading on floor mounted equipment. Seismic isolation can be used in addition to snubbers or can be made a part of the snubber.* While there are no standards for seismic snubbers, their capacities should be stated by the manufacturer and a rating is assigned by The Office of Statewide Health Planning and Development of the State of California. *Proper anchorage capacity including both horizontal shear and overturning uplift is required and a wedge anchor is recommended. Poured in place anchors are not feasible for snubber tie-down since equipment location is variable and* *often not defined specifically. Snubbers must be omnidirectional with at least a 3/8 inch resilient collar at least 4 snubbers must be used and all snubbers must be rated, Lama (1994).*

Nishio (1992) presents information of pipeline design in Japan. Figures 2 and 3 show pipe joint capacities for several Japanese pipe couplings. This excellent reference illustrates how the Japanese Gas Association provisions were developed and provides example calculations of their Deformability Index. The paper presents a discussion of the provisions and notes that the provisions use a value of deformation of 5.0 cm independent the liquefaction potential. Nishio introduces a probabilistic basis for assessing damage based on sample size. He shows the deformation capacity increases with diameter of the pipe.

In the analysis of continuous pipelines, it is possible to estimate the axial strain of the pipe in terms of the maximum ground strain:

$$\varepsilon_{p,\,max} = V_{max} \;/\; c_p$$

and the maximum curvature of the pipeline

$$\chi_{p,max} = A_{max} \;/\; c_s^2$$

where

A_{max}	Maximum ground acceleration
C_p	Compression wave propagation velocity
C_s	Shear wave propagation velocity
V_{max}	Maximum ground velocity

The maximum pipe joint displacement and joint rotation can be estimated by:

$$U_p = \varepsilon_{p,\,max}\, L$$

$$\theta_p = \chi_{p,max}\; L$$

where

L Length of pipe segment

Note that C_s and C_p can be estimated from G and ρ as follows:

$$C_s = \sqrt{\frac{G}{\rho}} \quad \text{and} \quad C_p = \sqrt{3C_s}$$

Eguchi et al. (1994) present an analysis of lifeline system damage which gives a good insight into the performance of pipelines. Table 1 presents relative performance of various types of pipe to shaking, liquefaction, landslide and fault rupture. They have compiled data on the number of repairs per 1000 feet of pipe and developed Figure 4 for fault rupture and ground shaking. The symbols are identified in Table 1. They note that the two mechanisms of ground displacement/fault rupture and shaking are different with the former being more damaging. Figure 5 shows their estimate of relative pipe performance under liquefaction and landslides conditions. Pipe diameter while a factor in pipe performance it was found that pipe material and joint type were more significant factors in normalizing field data. The data is intended to give system relative performance and not to be used to evaluate a single pipe Wang et al. (1992) illustrate use of flexible pipe joints, Figure 6.

The provisions of NAVFAC P355 Chapter 12 Section 12-7d pertain to design of essential pipelines and are part of this specification. They are as follows:

d. Seismic restraint provisions. Seismic restraints that are required for piping ..... will be designed in accordance with the following provisions.

(1) *General* The provisions of this paragraph apply to the following: ....

(b) Horizontal pipe. All horizontal pipes and attached valves. For the seismic analysis of horizontal pipes, the equivalent static force will be considered to act concurrently with the full dead load of the pipe, including contents.

(c) Connections. All connections and brackets for pipe will be designed to resist concurrent dead and equivalent static forces. The seismic forces will be determined from the appropriate provisions below. Supports will be provided at all pipe joints unless continuity is maintained. See paragraph (4) below for acceptable sway bracing details.

(d) Flexible couplings and expansion joints. Flexible couplings will be provided at the bottoms of risers for pipes larger than 3½ inches in diameter. Flexible couplings and expansion joints will be braced laterally unless such lateral bracing will interfere with the action of the flexible coupling or expansion joint. When pipes enter buildings, flexible couplings will be provided to allow for relative movement between soil and building.

(e) Spreaders. Spreaders will be provided at appropriate intervals to separate adjacent pipe lines unless the pipe spans and the clear distance between pipes are sufficient to prevent contact between the pipes during an earthquake.

(2) *Rigid and rigidly attached piping Systems.* Rigid and rigidly attached pipes will be designed in accordance with paragraph 12-3. The equivalent static lateral force is given by $F_p = ZI_pC_pW_p$ (SEAOC eq 1-10), where C_p is equal to 0.75 and is the weight of the pipes, the contents of the pipes, and the attachments. The forces will be distributed in proportion

to the weight of the pipes, contents, and attachments. A piping system is assumed rigid if the maximum period of vibration is 0.05 second (for pipes that are not rigid see paragraph (3) below). Figures 12-4, 12-5, and 12-6, (Shown in this report as Figures 7, 8 and 9) which are based on water-filled pipes with periods equal to 0.05 second, are to be used to determine the allowable span-diameter relationship for Zones 1, 2, 3, and 4 for standard (40S) pipe; extra strong (80S) pipe; Types K, L, and M copper tubing; and 85 red brass or SPS copper pipe.

(3) *Flexible piping Systems.* Piping systems that are not in accordance with the rigidity requirements of paragraph 12-7c(2) (i.e., period less than 0.05 second) will be considered to be flexible (i.e., period greater than 0.05 second). Flexible piping systems will be designed for seismic forces with consideration given to both the dynamic properties of the piping system and the building or structure in which it is placed. In lieu of a more detailed analysis, the equivalent static lateral force is given by $F = ZI_pA_pC_pW_p$ (eq 12-2), where A_p = 5.0, C = 0.75, and is the weight of the pipes, the contents of the pipes, and the attachments. The forces will be distributed in proportion to the weight of the pipes, contents, and attachments. Figure 12-7 (Shown in this report as Figure 10) may be used to determine maximum spans between lateral supports for flexible piping systems. The values are based on Zone 4 water-filled pipes with no attachments. If the weight of the attachments is greater than 10 percent of the weight of the pipe, the attachments will be separately braced, or substantiating calculations will be required. Temperature stresses have not been considered in Figure 12-7 (Figure 10 herein). If temperature stresses are appreciable, substantiating calculations will be required.

(a) *Use of Figure 12-7.* The maximum spans and design forces were developed for $ZI_pA_pC_p$ = 1.50. For lower $ZI_pA_pC_p$ values, the spans and forces may be adjusted by the values in Table 12-2. (Figure 12-7 and Table 12-2 are reproduced in this report as Figure 10)

(b) Separation between pipes. Separation will be a minimum of four times the calculated maximum displacement due to F_p, but *not less* than 4 inches clear between parallel pipes, unless spreaders are provided ...).

(c) Clearance. Clearance from walls or rigid elements will be a minimum of three times the calculated displacement due to F_p, but not less than 3 inches clear from rigid elements.

(4) Alternative method for flexible piping systems. If the provisions in the above paragraphs appear to be too severe for an economical design, alternative methods based on the rationale described in paragraph 12-4 and paragraph 12-8 may be applied to flexible piping systems.

Figure 11 shows acceptable details for sway bracing from NAVFAC P355.

NAVFAC P355 Chapter 14 has several figures which illustrate good engineering practice for pipelines. Figure 14-1 from NAVFAC P355 shows a sewer manhole in which the bell is located at the manhole and encased in concrete to increase its strength while still providing flexibility to the mating pipe. When a pipeline passes through a wall good practice allows a 2 foot square space in the wall around the pipe; the space is filled with oakum or other expandable material to provide for differential movement. Good practice provides flexible couplings at both ends of a 90 degree bend and on each of the three sides of a tee connection. The manual suggests that *prudent planning take into account the possible loss of electrical power to pumps and the potential need for manual operation of fuel pumps and backup lighting during an emergency.* A properly designed pipeline distribution system will include alternative routes and valves to isolate potential pipe breaks and maintain operation with improved reliability.

The following are taken from Chapter 14 and are required in this criteria:

- *No section of pipe in Zone 2, 3 or 4 shall be held fixed while an adjoining section is free to move, without provisions being made to relieve strains resulting from differential movement unless the pipe is shown to have sufficient stress capacity.*

- *When secondary or standby gas supply systems cannot be justified for a site, gas distribution networks for buildings in Zones 2, 3 and 4 housing essential functions dependent upon gas shall include an above ground valved and capped stub. Provision will be made for attachment of a portable, commercial-sized gas cylinder system to this stub*

- *For essential facilities in Seismic Zones 3 and 4, an earthquake activated gas shutoff valve shall be provided. If an earthquake activated shut-off valve presents the possibility of disrupted service in the buildings where the fire hazard is small, manually operated valves shall be installed.*

- *Buildings housing essential functions shall be provided with two or more water service lines connected to separate sections of the supply grid to minimize loss of service. Service shall be interconnected within the building by check valves to prevent backflow.*

- *Flexible connections shall be used between valves and lines for valve installation on pipes 3 inches or larger in diameter.*

- *Flexibility shall be provided by use of flexible joints or couplings on a buried pipe passing through different soils with widely different degrees of consolidation immediately adjacent to both sides of the surface separating the different soils.*

- *Flexibility shall be provided by use of flexible joints or couplings at all points that can be considered to act as anchors and at all points of abrupt change in direction and at all tees.*

- *NAVFAC P355 paragraph 12.-7* (cited above) *specifies restraints for critical piping in essential facilities.*

Piping containing hazardous materials shall contain numerous valves and check valves to minimize release of materials if there is a break. A secondary containment system should be incorporated where feasible. When piping is connected to equipment or tanks, use of braided flexible hoses is preferable to bellow-type flexible connectors since the latter has been noted to fail from metal fatigue. Welded joints are preferable to threaded or flanged joints. If flanged joints can not be avoided the use of self-energizing or spiral wound gaskets can allow a bolt to relax wile continuing to provide a seal, Association of Bay Area Governments (1990). Seismic shutoff valves should be used where necessary to control a system or process. These systems can be triggered by a mechanical sensor on the valve or by a remote electronic sensor which can control a number of elements. Choice of valves should be restricted to Navy approved valves to reduce leakage after closure. The Naval Facilities Engineering Service Center has tested seismic valves and authorized Navy public works and engineering field division can have access to that data.

New Tanks

Understanding the individual failure modes of individual tank components such as piping, restraints, tie down anchors, piles etc. is an important part of the development of a comprehensive design specification for quality performance. In general there are several types of tanks in use. Flat bottom vertical tanks vary in size from small 10,000 gallons to over several million gallons. Tanks can be anchored or unanchored. Large tanks can have internal columns to support the roof. Vertical tanks can be placed on a prepared mat or a ring foundation along the tank perimeter with or without edge confinement. Horizontal tanks are of cylindrical shape and are usually supported on two saddles. The typically range in size from 100 gallons to 10,000 gallons. Smaller tanks can be supported on legs. Typically fuel tanks for portable generators are of this type. Summers (1997) lists major causes of tank failure and includes the following:

- Buckling of the tank wall (termed elephant foot buckling)
- Breakage of inlet/outlet piping from uplift
- Tearing of the tank wall at discontinuities
- Tearing of tank wall from overconstrained stairways between the foundation and tank shell
- Roof damage caused by sloshing
- Foundation failures and liquefaction

Schiff (1991) presents a summary of observed damage to tanks. Flatbottom vertical tanks tend to be most vulnerable to earthquakes, especially tanks with a large liquid depth to radius ratio. One failure mode of the tank is buckling and is caused by rocking of the tank or differential settlement of the foundation under the tank Unanchored tanks with a radius-to-wall thickness of over 600 have been damage most often. These tanks develop sufficient overturning moment to cause the edge to lift off the ground. The opposite side sustains high compressive stresses which cause bulging at the base. Summers (1997) reports that a 100-foot diameter tank 30 feet high sustained 14 inches of uplift in the 1971 San Fernando earthquake. Similar tank behavior was noted in the 1989 Loma Prieta earthquake with an observation of 6 to 8 inches of uplift and 18 inches in the 1964 Alaska earthquake. Even anchored tanks can fail in this manner if there is anchorage failure. *Generally a pattern of well distributed anchor bolts works best compared with* *fewer larger bolts. Maintenance is requirement to inspect the condition of the anchor bolts and replace those with corrosion.*

Vertical motion can cause local tensile membrane deformation, elephant foot bulging, at the base of the tank. This can also be induced by rocking. It is interesting to note the annular volume of the bulge is about equal to the earthquake vertical displacement times the tank area. It is postulated that the fluid has high inertial and the increase in fluid pressure from the vertical component of the earthquake causes the perfectly symmetrical bulge. Increasing the wall thickness may reduce the occurrence, but might simply result in the buckling occurring high up on the tank. The weld between the tank base plate and side wall has also been observed to fail. This is caused by uplift forces and is often associated with corrosion induced weakness. A failure of the weld can open a portion of the seam causing rapid loss of contents and a partial vacuum in the tank causing internal buckling. *Tank venting is important to restrict implosion.*

Small unanchored tanks less than 30 feet in diameter have been observed to slide on their foundation. In tanks which are full, sloshing can cause roof and upper wall failures. As noted liquefaction was a major cause of the extent of waterfront damage and can cause settlement and lateral spreading.

The primary failure mode of horizontal tanks is anchorage failure or inadequate anchorage which causes tank slippage off the saddles. Typically the tank is fully anchored only on one side to allow for expansion. The single restraint must be capable of withstanding horizontal, vertical and torsional components of motion. The saddle must be designed to resist forces acting on its weak axis as well. Elevated fuel tanks often fail by buckling of the supports. These tanks stands require adequate tie-down and diagonal bracing

Water tanks tend to be kept full however hydrocarbon tanks tend to be half-full and sloshing must be considered. Lack of tank venting has resulted in implosion. Anchor bolts embedded in concrete used for tank uplift restraining must have sufficient concrete confinement to prevent pullout. Shear reinforcing should be used to provided needed concrete confinement to prevent anchor bolt failure. *Typically anchor bolts for new construction are designed with a safety factor of 4; a value of 3.0 is used for evaluation of existing anchors. Provisions must be made to evaluate the effect of corrosion in reducing the strength of existing construction.*

To achieve the required system performance and satisfy regulations additional backup hazardous material containment systems are used. Containment systems are composed of either a singular system or a dual system as mandated by public law discussed in the Criteria. A singular system provides only a single structural element system for material containment. Singular systems are restricted to small systems of less than 660 gallons such that a failure shall not produce catastrophic damage. A dual system is composed of a primary containment structure and a secondary containment system which shall function should the primary system be damaged. Containment systems open to rain shall need to be drained.

Design of tanks shall utilize the API 650 procedures discussed above.

Tanks shall be designed against sliding and uplift and be fully anchored. The height of sloshing may be calculated using an equation by Wozniak and Mitchel (1978). This height should be used for freeboard calculations associated with roof damage. The hydrodynamic forces which create overturning moments also act on the foundation and must be taken into account in foundation design.

Essential tanks shall be designed to resist Level 2 earthquakes (10 percent probability of exceedance in 100 years) using response spectra and the API 650 procedures.

For both ordinary and essential tanks, a requirement exists to prevent uncontrolled loss of contents and pollution of the environment an event with a 10 percent probability of exceedance in 250 years. Such requirements shall be met by provision of a containment system. Singular systems must be designed so that the structure itself provides the margin of safety to preclude release of materials. Dual systems may be evaluated on the basis of total system performance allowing for the presence of the secondary confinement, such that any release from the primary containment is confined within the secondary containment. The secondary containment must function at such a level so as not to permit an unacceptable release of materials.

Failure of pipe to tank connections is common when there is insufficient flexibility to accommodate differential motion between the tank and pipe network. This can be prevented by having the first pipe anchor point at a sufficient distance (15 pipe diameters minimum) from the edge of the tank and the pipe oriented in a radial direction away from the tank. Flexible connections and expansion joints can accommodate differential motion provided they are sized properly. The most important element of seismic design of pipelines is proper siting. It is imperative to avoid areas of lateral spreading and landslide. *Additionally stairways should not be attached to both the foundation and the tank wall.*

Summers (1997) presents the following information:

> Tank uplift during earthquakes can damage attached piping and other appurtenances. ...... anchored tank appurtenances may he designed for some level of anchor bolt stretch. A value of 2 inches is proposed in the latest NEHRP provisions (BSSC, 1994).
>
> API 650 states that piping attached to the tank bottom that is not free to move vertically shall be placed a radial distance from the shell/bottom connection of 12 inches greater than the uplift length predicted by the API 650 uplift model. The API 650 uplift model, however, may underpredict the amount of radial uplift (Manos, 1986; Dowling and Summers, 1993). It may be prudent to consider changing this requirement to... twice the API 650 model.......
>
> Walkways between tanks should he designed to accommodate relative movement of the tanks. ..... In lieu of a more rigorous analysis, a walkway should he designed to accommodate a total of 12 to 18 inches of movement, at least in the zones of high seismicity.

Attached ringwalls should be designed appropriately. Anchoring a tank to a small ringwall and not developing the forces into the soil by the weight of the ringwall or with piles should he viewed with caution. Anchor bolts need to be designed such that they behave in a ductile manner, both in terms of the force transfer to the shell and pullout from the concrete foundation.

Existing Tanks

Tanks built prior to the late 1970s probably lack consideration of seismic design since it was during that period in which code provisions were first implemented. However the provisions in API and AWWA are generally thought to be conservative such that evaluation of existing tanks by the new tank criteria may unduly penalize them. Summers (1997) reports the following:

> There are several alternatives to the API methodology that might be considered for use in evaluation of existing tanks. One such method is a modified version (Dowling and Summers, 1993; Summers and Hults, 1994; and ASCE Task Committee on Seismic Evaluation and Design of Petrochemical Facilities, 1997) of a method developed by George Manos (Manos, 1986) presented herein Manos' method is based on experimental studies, as well as on observed behavior of unanchored tanks during past earthquakes. Instead of trying to model the complex uplifting plate behavior, Manos assumes a stress distribution at which the shell will buckle and solves for the resisting moment produced by the sum of the stresses. This resisting moment can then be compared to the overturning moment and the resisting acceleration solved for.
>
> The method proposed herein for evaluation of unanchored storage tanks is based on that of Manos, but includes some important variations. The most notable of these are (Dowling and Summers, 1993):
>
> a. Tank anchorage is recommended in zones of high seismicity whenever the ratio of safe operating height to tank diameter exceeds two. Based on the data presented in Manos, and the higher level of risk for taller tanks, this is believed to be the upper limit of applicability of the Manos method.
>
> b. The allowable compressive stress in the tank shell should not exceed 75% of the theoretical buckling stress, as presented in Manos, nor should it exceed the material yield strength. This last requirement is significant for thicker-walled tanks. Note that an increase in the allowable compressive stress beyond 75% of the theoretical buckling stress may be justified under certain circumstances.
>
> c. The compressive force in the tank shell should not exceed the total weight of the fluid contents. This has the effect of imposing an upper bound on the resisting moment.

A comparison of the results of an evaluation of a 35 ft diameter, 30 ft high tank in a high seismic zone filled to a height of 26 ft 4 in, using the modified Manos and API methodologies, ... (was made. The) API approach would require either a reduction in fill height by about 40% to 16 ft 6 in or tank anchorage, whereas the modified Manos method indicates that the seismic safe operating height can be increased to 20 ft 1 in. Hence, the required reduction is reduced from 9 ft 10 in to 6 ft 3 in, and the benefit is immediately apparent.

The Manos (1986) develops the following relationships for the compressive member stress distribution near the tank bottom as:

$$\sigma_{max} = 0.75\ \sigma_{cl}$$

where

$$\sigma_{cl} = \frac{E\,t_s}{R\sqrt{3(1-\upsilon^2)}}$$

$$\sigma = \sigma_{max}\cos\left(\frac{\pi\phi}{2\phi_0}\right)$$

if $\upsilon = 0.3$

$$\sigma = 0.46\frac{Et_s}{R}\cos\left(\frac{\pi\phi}{2\phi_0}\right)$$

$$\phi_0 = 0.65\ S\left(\frac{R}{H}\right)^n\left(\frac{t_s}{t_p}\right)^{0.1}$$

$$n = 0.1 + 0.2\frac{H}{R} \le 0.25$$

where

E	Young's modulus
H	Liquid height
R	Tank radius
S	Foundation coefficient
t_s	tank wall thickness
t_p	tank bottom plate thickness
υ	Poisson's ratio
σ	axial member stress
σ_{cl}	Theoretical buckling stress

Manos develops an empirical expression for the limiting impulsive acceleration capacity of a tank, C_{eq}, as

$$C_{eq} = \frac{0.372}{\rho_w} \frac{S\,E\,t_s^2}{G\,R\,H^2} \frac{m_t}{m_1} \left(\frac{R}{H}\right)^n \left(\frac{t_s}{t_p}\right)^{0.1}$$

where

G	Liquid density ratio
m_1	Liquid impulsive mass
m_t	Total liquid mass
ρ_w	Density of water or tank liquid

Manos compares the acceleration capacity to the applied acceleration which is based on a tank response spectrum determined from an amplified ground motion spectrum between the periods of 2 and 9 seconds having 2 percent damping. He proposes a 4.3 acceleration amplification factor to be applied to the ground motion spectrum as a conservative approximation of structure amplification. The acceleration capacity must be larger than the acceleration demand.

UTILITIES ON PIERS

Piers may contain pipelines for freshwater, saltwater, steam, compressed air, waste oil, sewer, fuels, as well as electrical power and communication lines. Ship demands dictate the configuration. In general design of these lines follows the general provisions discussed herein. It is essential that the lines be attached to the supporting structure with sufficient rigidity that the lines are restrained against independent movement. Attachments to a pier may be analyzed as simple two-degree-of freedom systems as discussed in NAVFAC P355, Chapter 12. Resonance amplification can occur when the natural period of the supported pipe is close to the fundamental period of the pier structure. Flexible connections/sections shall be used to bridge across expansion joints or other locations where needed. All piping and utility lines on a pier shall be designed as essential construction. Specifically, the provisions of NAVFAC P355 Section 12-7d shall be used. Section 12-7d is discussed above under pipelines. Pipelines containing hazardous materials may have to be of double wall construction based on requirements of local environmental requirements. Check-valves should be used to minimize the loss of contents to minimize the size of a spill if there is a pipeline break.

ELECTRIC POWER

A typical electric power system includes generating plants, transformer and switching substations, distribution lines, local transformers and backup generators. Linkages exist between electric power and other lifelines; for example, electricity is needed for pumps to maintain pressure in water distribution systems. The Navy suffered a major outage to its generating plant during the Guam earthquake. The plant is situated on soil which is liquefiable. The majority of the plant is constructed on piles which withstood the ground shaking. The cooling water intake to the plant was not supported on piles and settled significantly fracturing the intake line and forcing an extensive closure of the plant. Most civilian power station damage has resulted from overturning or sliding of inadequately anchored or braced components. Often electrical equipment is situated on top of poles or supports undergoes extensive displacements rupturing attached cables. Pole mounted transformers are supported on raised platforms; typically they are not secured to the platform.

Large power transformers are critical elements in a power distribution system. They are often special order items and not easily replaced. Bushings made of ceramic materials provide a way for the electrical conductor to enter the case of the transformer. Typically the bushings are held in place by a spring loaded internal tension member which holds it against the transformer case. A seal between the bushing and the housing contains nitrogen pressure. Failure of this seal can cause oil to leak out and present a potential for fire. A transformer radiator is an assemblage of pipes attached to the transformer to cool the internal oil. Dynamic response of the radiator can result in support damage and leaks

Inadequately mounted transformers have been observed to fall from pedestals causing major damage to bushings, radiators and internal parts. An alternative failure mode is excessive sliding without overturning. Sliding breaks the rigid bus connections, the lightning arrestors and insulators. Past practice had transformers mounted on rails anchored in concrete slabs. When these mounts failed, extensive damage resulted. Schiff (1980) suggests that new installation design use concrete pads with steel anchor plates securely embedded in the pad and flush with the surface. The transformer is welded to the anchor plate eliminating the need for an intricate pattern of tie down bolts. Spare transformers were kept unsecured for relocation as needed, which can overturn. All transformers whether in service or spare require the same restraint.

A linetrap sits on top of a ceramic insulator supported by a tubular steel column. The steel column is generally welded to a base plate which is bolted to a concrete pad. The tall flexible inverted pendulum can under go large displacements and induce high moments in the anchorage, base plate welds have been observed to fail. The linetrap is suspended between other components such as a circuit breaker and a disconnect. Lateral motions of 4 inches are typical during and earthquake and will fail cable bushings if sufficient slack is not provided in the cables. Excessive slack can result in short circuit loads when conductors touch grounded portions of the structure. When designing flexible mounted equipment it is necessary to estimate the maximum relative displacement between the two ends of the cable. A computer program is available to perform this simple calculation using a number of earthquake records, Figure 12. Both pieces of equipment should be analyzed as single degree of freedom approximations and maximum response determined for a variety of earthquakes. An alternative is to provide breakaway connectors which will disengage under lateral load and which can be easily reconnected.

Criteria for electrical power lifelines focuses on providing adequate anchorage. All transformers on poles or platforms shall be anchored against overturning or sliding. All *equipment shall be anchored as required. Equipment deemed as of ordinary importance shall have lateral force requirements based on provisions of the 1994 Uniform Building Code and NAVFAC P355. Equipment deemed as essential shall have the lateral force requirement based on local site conditions using peak ground acceleration for essential level facilities (10 percent exceedance in 100 years) and a response spectra. In any case lateral forces shall not be less than Code provisions with an importance factor for essential structures/components . This resulting force shall be used as a substitute for Code forces and all remaining Code provisions will apply.*

Special shore power is required for nuclear submarines and is discussed in MIL-HDBK-1025/2A section 3.8.7.1. Specifically there is a need for standby power with a maximum down time of 5 minutes.

TELECOMMUNICATIONS LIFELINES

Telecommunications encompasses conventional telephone requirements, communication transmitters and receivers in conjunction with airfield operations, and military requirements in support of operational readiness. Systems include buildings with electronic components, large computers on raised floors, cables in racks, batteries on shelves, conventional telephone lines, microwave antenna, satellite communications, towers for antenna, etc. As can be noted, there is a diversity of requirements; yet, all of the elements have a commonality. First *the equipment must be rugged enough to withstand the shaking. The IEEE has established fragility requirements for some equipment found in nuclear power plants.* Some other types of equipment also have fragility data. Second the *equipment must be attached in a manner to prevent damage. Attachment can be made by rigidly securing the item against overturning and sliding or where the equipment is delicate it may be mounted on isolators to reduce transmitted motions. A variation of both approaches consists of leaving a large piece of equipment free to slide within restrained limits.* This limits the shaking motion which can be transmitted to the equipment by allowing sliding to occur; elastic bumpers limit the range of motion. Obviously the equipment must have an aspect ratio to preclude overturning.

Traditional damage has included overturning of cabinet mounted electronics, failures of battery racks, failures of suspended ceilings, rupture of piping and water damage to equipment, rupture of cables connecting equipment which became dislodged, weld failures, and inadequate sizing of restraints. Design calculations must consider the inertia force of an object in overturning and sliding. Elements attached to the structure must consider the relative displacement between anchorage points. Flexible supports must consider resonance points where the period of vibration of the flexible mount is the same as that of the structure; stiffening the mount can eliminate resonance.

Raised floors are a common occurrence in computer centers. They serve as a plenum for air conditioning and a means for routing under floor cables. Generally the raised floor system consists of supporting pedestals, stringers and removable 2-foot by 2-foot floor panes. When the raised floor is confined by surrounding walls performance is enhanced. *For best seismic resistance the pedestals should be considered as cantilever columns and anchored into the supporting concrete* *slab. Diagonal bracing is available and can be used for additional restraint. The stringers should be firmly attached to the pedestals for added bracing.*

Required support equipment generally includes backup power generators, uninterruptible power supplies, emergency lighting, voltage controllers, etc. and may also include air-conditioning units, halon firefighting systems etc.

CALCULATION OF LATERAL FORCE REQUIREMENTS

The Uniform Building Code gives an equation for the total lateral force on elements of structures, nonstructural elements and equipment supported by structures as:

$$F_p = Z\, I_p\, C_p\, W_p$$

where

C_P	Numerical coefficient based on type of component (range of values 0.75 to 2.0)
I_P	Importance factor
W_P	Weight of element or component
Z	Zone factor

Note I_p equals 1.5 for elements of essential facilities and hazardous facilities per Table 16K of the Code. This represents an increase from the 1.25 used for essential buildings.

The above equation is to be used to calculate the force in connectors. The minimum design lateral forces are at a service level rather than at ultimate. Provision is made for combination of loads. The coefficient C_P is for elements and components and for rigid and rigidly supported equipment, Table 2. Rigid or rigidly supported equipment is defined as having a fundamental period less than or equal to 0.06 seconds. The lateral forces of a nonrigid piece of equipment above grade having a period greater than 0.06 seconds are to be determined by a dynamic analysis taking into account the properties of the structure and the equipment. In lieu of a dynamic analysis, the values of C_P may be doubled up to a limit of 2.0. The value of C_P for elements, components, and equipment laterally self-supported at or below ground level mat be 2/3 of the table values offset forth in the code but not less than those determined by treating the item as an independent nonbuilding structure which for a rigid structure would be:

$$V = 0.5\, Z\, I\, W$$

Structures which support flexible nonstructural elements in Seismic Zones 3 and 4 whose weight exceeds 25 percent of the weight of the structure must be considering the dynamic interaction of the two.

Structures specified as nonbuildings will be designed using the general building equation:

$$V = [(ZIC) / R_w]* W$$

$$C = \frac{1.25S}{T^{2/3}}$$

where

Z	Zone factor
I	Importance factor = 1.0 for ordinary or 1.25 for essential
S	Soil factor
T	Period of structure
R_w	Response modification factor or ductility factor

and values of R_w are from Table 3.

The Uniform Building Code requires raised floors in seismic zones 2, 3, and 4 to be designed for the floor loads using the above equation for F_p. The coefficient C_p is set at 0.75 and W_p equals the floor dead load plus 25 percent of the floor live load plus a 10 lbs/sq ft partition factor. Heavy air conditioning units located in the same room with equipment on raised floors may need special design requiring a steel supporting platform separate from the raised floor.

The 1994 NEHRP provisions calculate seismic forces as:

$$F_p = 4.0\ C_a\ I_p\ W_p$$

Alternatively F_p may be calculated by:

$$F_p = [a_p\ A_p\ C_a\ I_p\ W_p] / R_p$$

$$A_p = C_a + (A_r - C_a) (x / h)$$

$$A_r = 2.0\ A_s \leq 4.0\ C_a$$

$$F_{p\ minimum} = 0.5\ C_a\ I_p\ W_p$$

where:

F_p Seismic design force centered at the component's center of gravity and distributed relative to component's mass distribution.

a_p Component amplification factor that varies from 1.00 to 2.50 (select appropriate value from FEMA 223A Table 3.2.2 or Table 3.3.2).

A_p Component acceleration coefficient (expressed as a fraction of gravity) at point of attachment to structure.

I_p Component importance factor that is either 1.00 or 1.50

W_p Component operating weight.

R_p Component response modification factor that varies from 1.50 to 6.00 (select appropriate value from FEMA 223A Table 3.2.2 or Table 3.3.2).

C_a Seismic coefficient at grade as determined expressed as a fraction of gravity

A_r Component acceleration coefficient (expressed as a fraction of gravity) at structure roof level.

x Elevation in structure of component relative to grade elevation.

h Average of elevation of structure relative to grade elevation.

A_s Structure response acceleration coefficient (expressed as a fraction of gravity):

$$A_s = 1.2\,C_v / T^{2/3} \leq 2.5\,C_a$$

Note that A_s shall be calculated for each principle horizontal direction of the structure. The largest value for A_s shall be utilized in determining A_r.

C_v Seismic coefficient as determined from FEMA 223A Sec. 1.4.2.3 or Table 1.4.2.4b (expressed as a fraction of gravity).

T Effective fundamental period of the structure as determined in FEMA 223A Sec. 2.4.2.1 and 2.4.3.1.

The force F_p shall be applied independently vertically, longitudinally, and laterally in combination with service loads associated with the component. When positive and negative wind loads exceed F_p for nonbearing exterior wall, these wind loads shall govern the design. Similarly when the building code horizontal loads exceed for interior partitions, these building code loads

LIFE CYCLE COST ANALYSIS

Life cycle cost analysis shall be performed using the provisions in NAVFAC P355.2 Chapter 7 and the guidance in NCEL Technical Notes N 1640 and N 1671. Economic analysis may be used to study alternative concepts provided a release of hazardous materials under the Level 2 earthquake shall not be permitted.

Previous work, Ferritto (1981, 1982, 1983, 1984a, 1984b) presents a detailed procedure for evaluating the economics of seismic design. This work has been adopted in P355.2 Chapter 7 prescribing techniques for economic analysis. In general the costs of seismic strengthening are determined as a function of the design level. Damage functions are developed which quantify the expected damage as a function of the design level and the level of applied seismic ground motion. A probabilistic distribution of ground motion is used to compute expected damage over the range of damage potential. The expected damage is computed in terms of present worth. Functions can be included for the loss of use of the structure and life loss potential. Such a technique can be applied to the evaluation of the most effective design level. The procedure is site specific because the seismic exposure varies with the specifics of potential seismic sources and their distance from the wharf site, although general conclusions can be made for a typical waterfront site with the absence of a local dominant fault. Additional factors beyond economics also must be considered. These include:

- Importance of the facility, duplication of the capability etc.
- Disaster preparedness and recovery capabilities.
- A general seismic risk strategy, financial capacity and risk tolerance.

RELIABILITY

Reliability is a measure of the confidence in the ability to achieve the stated structural performance goals during its service life. While two structures may both achieve the same end function, the variance measures the spread in the safety margin. A structure with low variance has a narrow area of uncertainty. Seismic design inherently carries high levels of uncertainty associated with the load producing mechanism- the earthquake. No two earthquakes are alike. The earthquake process represents the rupture of a unique fault system under a unique stress pattern at a specific time in history. The loading process is composed of the source mechanism, the path from hypocenter to the site, and the local site effects which can amplify ground motion significantly especially at the waterfront. Added to this are the mechanisms of soil-structure interaction and inelastic dynamic structural response. Each of these five elements is a significant problem in itself which we can not solve but only approximate by procedures of various depth of modeling and with various levels of fidelity. There are major uncertainties such as:

- The size of the earthquake which is possible- this may be expressed as the most probable event or a specific deterministic design event or a maximum size event.

- The location of the event relative to the site.

- The ground motion produced at the source which is attenuated to ground motion at the site. Ground motion may be expressed as acceleration or velocity, as some peak value or as a time history or as a spectra. Each has major bands of uncertainty

- The amplification of site bedrock motion to the surface.

- The composition of the earthquake motion as compressional, shear, Love and Rayleigh waves.

- The characterization of the structure in terms of components including development of a structural model.

- The characterization of soil and structure material response and development of adequate constitutive models to be used in analysis

- The ability to predict large deformation inelastic structural response.

There are certain engineering calculations which we can perform better than others especially when we have to extrapolate future behavior based on a finite record of previous history. We can predict the level of site ground acceleration more accurately having a return time of 100 years than that of 1000 years. We can predict structural stresses more accurately if the structure remains within its elastic response range than if it undergoes collapse-level inelastic large deformations. Any criteria which we develop would be more reliable if it focused on the things we do well rather than the things we do poorly. We could establish a design level such that a structure would remain elastic under a modest load. This would establish the structural member sizes required to achieve the design requirement. We could achieve the same selection of member sizes by specification of a higher less probable loading and some associated levels of inelastic allowable ductility. Both designs produce the same member sizes. One is easier to accomplish and has a higher reliability associated with it. Thus the criteria should if possible peg the design on modest levels of ground motion and elastic response rather than a 1000-year ground motion and collapse level response. It is more difficult to predict the rare motion and the collapse of the structure than it is to predict an elastic condition under modest motion levels. When we establish a criteria using a two earthquake criteria it would be more reliable for the lower level event to control the design and the higher level to be used as insurance against collapse. This is not always possible. Local seismicity conditions in Los Angeles at the Port of Los Angeles allow the design to be controlled by the lower event. The Port of Oakland is strongly influenced by the close-by Hayward fault and the controlling event is the larger collapse event.

The seismicity procedures developed by the Navy, Ferritto (1994) can compute the probability distribution of site acceleration and the associated uncertainty. We can estimate structure failure in terms of ultimate section capacities and associated inelastic ductilities and drifts. Conceptually we can express the probability of an unacceptable outcome such as structure collapse or unacceptable performance. The overlap of loading and capacity represents failure. As we minimize the overlap we increase the probability of acceptable performance and meeting the positive goal. As our knowledge increases the width of the distributions becomes less so that we have less uncertainty associated with the load or the strength. As the distributions narrow we get less overlap assuming the means remain constant.

BASE LIFELINE ASSESSMENT TEAM

The Office of Secretary of Defense (OSD) has established a secure energy policy which is applicable to earthquake lifelines as well.

> "It is Department of Defense policy to have an adequate, secure energy supply to meet operational mission support requirements. The goals of the Defense Energy Security Program are: (1) to evaluate the ability of Defense installations to maintain the level of energy supply necessary to provide mission support; and (2) correct deficiencies that could jeopardize this energy supply.

A procedure was established to implement the OSD policy through an Energy Security Plan which is a comprehensive plan addressing all aspects of maintaining facility energy security. The Energy Security Plan is a focused study on the relationship of the energy supply system to facility mission requirements. Step 1 of the Plan identifies criteria for determining critical functions that require secure energy and selecting those facilities supporting the required mission functions. Step 2 is the development and application of an assessment procedure. Step 3 calls for initial assessments and follow-on assessments every 2 years The assessment is to identify the energy security deficiencies, their severity, and recommended options for corrective action. Step 4 identifies the resource and funding process for the corrective action.

Base post-earthquake recovery efforts must focus on life safety, disaster control and sustaining required operation of essential functions. Recovery efforts must be prioritized to maintain required operations and minimize further damage. It is also essential to relate the repair of utility systems to mission needs since experience has shown that utility system disruptions can produce major impacts upon mission essential base functions. NCEL TR920U, Base Vulnerability Assessment Guide, Chapter 10 presents a procedure for evaluating utility vulnerability which is applicable to lifeline analysis.

The elements of the base lifeline assessment procedure are to:

- Form an Assessment/Mitigation Team
- Gather information about lifeline utilities
- Determine essential-function requirements from users
- Consider utility outage scenarios
- Assess the vulnerability of required utility lifelines
- Develop mitigation measures

The most significant step in conducting a lifeline assessment is understanding what are the critical operations of the base and tenant activities (ship building and repair, communications)? What critical facilities support these operations (dry-docks, computer and communication center, industrial facilities) and what utilities are critical to support the critical operations? In addition to mission requirements are the disaster control and recovery functions which are also highly dependent on lifelines.

The utility/lifeline assessment team must contain a technologist with the ability to understand and assess the base's utilities. The working group should include a representative of the utilities section of the Public Works Office, and a representative of the regional Naval Facilities Engineering Command Field Division. This group should actively meet with representatives from the base departments and tenant commands to develop an understanding of operational requirements. The discussion with base representatives and tenant command representatives will provide much information based on the breadth of experience and knowledge of these activities. The team will also need accurate utility system drawings and diagrams for each system being evaluated. Operators and maintenance personnel can often identify vulnerable points for critical systems. Reports from utility studies, communication system studies, utility contingency plans, security and other plans and studies may provide drawings and analysis that are applicable to this effort. Key hardware components servicing each essential operation need to be identified. Key hardware components are pieces of equipment which must remain operational to support the operation and represent critical links. Utility outage scenarios must be developed based on the seismic potential and the key hardware components accessibility and fragility. The effects of the utility outages must be analyzed including the evaluation of the response, repair, and recovery capability of the base. Once the team has accomplished the utility systems assessment, they can identify the corrective measures necessary to remedy deficiencies. These remedies are likely to include projects and/or activity operational procedures. The team should rank the mitigation measures and develop plans to implement them.

Analysis

The approach outlined in this section is a guide to assessing the vulnerability of those utilities which provide service to essential facilities. The survey team should examine electricity, water, thermal (steam, hot water, chilled water, natural gas), sanitary and industrial wastewater, compressed air, and communications as well as any other utility. The information gathered is meant to stimulate the thought process and is a tool to assess the utility systems. In gathering these data one will not only compile a comprehensive source of valuable utility information, but will also discover information gaps which might prove critical in an actual utility outage situation. The vulnerability assessment report produced from this information will require rigorous analysis of the particular lifeline utility systems and operating procedures. In analyzing a utility system, it is best to follow a logical pattern from the point of utility supply through the onbase distribution system to the final end-user. In the assessments, provide simplified schematics of each utility system which have key components.

For each outage scenario considered, both area-wide and local utility outages, the impact upon the essential operations should be assessed, the minimum utility requirements needed to support these operations should be determined, and possible corrective measures to meet these requirements should be evaluated. Consideration should be made regarding the interrelationships between utility systems.

Response, Repair, and Recovery Capability

For every key component identified, a comprehensive evaluation of Public Works Department's ability to restore the item to affected areas should be conducted. For example, restoration of power may include repair to the disabled component, replacement of the disabled component, actual bypass of that component or provision of backup power. The potential for subsequent problems from the repair, replacement, or bypass should be part of the overall evaluation. Items that should be considered include the following:

1. Availability of spares and replacement equipment (location, time, administrative procedures)
    a. Onsite
    b. Commercial utility
    c. Commercial suppliers
    d. Other sources

2. Availability of equipment needed to effect repairs

    a. Heavy transport equipment
    b. Trucks
    c. Special tools and machinery

3. Availability of personnel

    a. Activity PWD or PWC

        1. Military/civilian mix
        2. Level of training
        3. Degree of experience
        4. Knowledge of installation systems
        5. Multiple assignments and responsibilities
        6. Staff reassignment

    b. Commercial contractors

        1. Number of contractors available and past relationships
        2. Formal agreements or contracts
        3. Administrative and financial limitations

    c. Commercial utility

1. Availability potential
2. Formal and informal agreements
3. Other commitments or obligations

4. Consideration of adverse working conditions and circumstances

5. Implementation of load shedding/conservation to match available supply

6. Availability of backup generator sets

   a. Verification of operation of generators
   b. Maintainability/ability to operate for extended periods
   c. Refueling requirements and procedures
   d. Ability to relocate and connect to loads

Key Hardware Components

The following sections are based on NCEL TR920U, Chapter 10.

Electrical Distribution System

a. Commercial feeds to activity and their point of origin, point of connection to base, and capacities
b. Primary onbase generation
c. Backup generators

   1. Number of fixed and portable sets assigned
   2. Other generators available on the installation
   3. Size, age, and present assignments

d. All mission-essential loads and all key components utilizing a current wiring diagram
e. Load shedding and any other relevant contingency plans

Water Distribution Systems

Water system key hardware components must include potable water, fire protection water and water requirements related to thermal energy systems (makeup water for boilers and cooling towers).

a. All mission essential loads and key components shown on current drawings
b. Commercial lines serving the activity and the points of origin
c. Onsite water sources (well, desalination, etc.)
d. Onsite water treatment facilities

e. Capacity and location of storage facilities
f. Key components dependent on electrical power

    1. Pumps
    2. Water treatment equipment
    3. Valves
    4. Controls

g. Backup electrical generator sets

    1. Number of fixed and portable sets assigned
    2. Size, age, and present assignments

h. Availability of onbase/commercial water transport
i. Treatment chemical requirements

Wastewater (Sewage/Industrial Systems)

Wastewater system key hardware components should include collection and treatment facilities for domestic sewage and industrial wastewater. Consider the possible creation of hazards associated with mixing incompatible industrial wastewater streams (i.e., acid wastewater mixed with cyanide wastewater create toxic hydrogen-cyanide gases). Additionally, repairs to sanitary sewers, where waste may be septic, should be accomplished with protective gear and respiratory equipment.

a. Public lines serving the activity and their point of origin
b. Generating activities
c. Onsite treatment facilities
d. Storage capacity (i.e., 55 gallon drums, tankers, holding tanks, etc.)
e. Critical functions which generate wastewater
f. Key hardware components from point of generation to treatment/disposal
g. Key components dependent on electrical power

    1. Pumps
    2. Wastewater treatment equipment
    3. Valves
    4. Controls

h. Backup electrical generator sets

    1. Number of fixed generator sets
    2. Size, age and present assignment

i. Availability of base/commercial transport (i.e., tankers, barges etc.)
j. Treatment chemical requirements

Compressed Air

a. Mission-essential functions requiring compressed air
b. Sources of compressed air (central, point of use)
c. Key components dependent on electric power
d. Identify compressors capable of operating independent of electrical power
e. Isolation valves
f. Cross connects between distribution lines
g. Backup compressors at critical points of use
h. Backup electrical generator sets

Thermal Energy Systems

Hardware and system components included a thermal energy network can be quite numerous and varied. Oil, natural gas, coal, propane, and other fuels may be used to fire boilers. Central boiler plants for provision of steam or hot water, small individual boilers, central plants for chilled water, and the other configurations of a thermal energy distribution system are possible.

a. Mission-essential functions that require thermal energy (steam, hot water and chilled water)
b. Number and locations of fuel delivery routes

    1. Key valve location and function
    2. Standby power for fuel delivery systems

c. Number and locations of on-site fuel sources
d. Number and locations of fuel storage facilities

    1. Above or below ground
    2. Central or dispersed storage
    3. Storage tank isolation features

e. Number and location of thermal energy conversion facilities

    1. Steam and hot water boilers

        a. Capacity
        b. Pressure
        c. Dual fuel capability

    2. Chilled water generators
    3. Backup thermal conversion systems (MUSE boiler)

f. Number and locations of thermal energy distribution systems

1. Distribution pipelines
2. Key valve location and function
3. Standby power for product distribution

g. Key components requiring electric power

1. Standby power for fuel delivery Systems
2. Boiler plant (controls, fans, pollution control devices, etc.)

CHECKLIST FOR WALK-THROUGH SCREENING

The Army Corps of Engineers sponsored a study of lifelines on military bases. One of the products which came out of that work was a set of checklists for screening water supply lifelines part of which is reproduced below.

Pump Stations

T F **PIPING PENETRATIONS:** Piping at wall penetrations and equipment has flexible connections or sufficient clearance.

T F **ANCHORAGE:** Pumps, motors, control cabinets, generators and telemetry controls are adequately anchored.

T F **VIBRATION ISOLATORS:** Vibration isolated pump and drive units have seismic snubbers to limit motions.

T F **OFF-SITE POWER:** Off-site electrical power or telemetry has backup provisions.

Process Tanks And Structures

T F **ANCHORAGE:** Tanks are adequately anchored.

T F **IMMERSED COMPONENTS:** Concrete or timber baffles, rotating equipment, and other immersed components have been designed for sloshing and inertial effects.

T F **EMERGENCY CHLORINATION:** The plant can bypass treatment and provide emergency chlorination if damaged by an earthquake.

T F **PIPING PENETRATIONS**: Tanks have flexible connections at piping penetrations.

T F **LIQUEFACTION**: Structures are not buried in liquefiable soil.

Equipment and Piping

T F **ANCHORAGE:** Plant equipment is adequately anchored.

T F **COMMON FOUNDATIONS:** Pumps and motors are on common foundations.

T F **PIPING CONNECTIONS:** Flexible piping connections are used on all equipment.

T F **EXPANSION JOINTS:** Piping which crosses expansion joints has flexible connections.

T F **BRACING:** All piping runs of 4 inches diameter and larger are transversely and laterally braced.

T F **HAZARDOUS MATERIALS PIPING:** All piping runs of 1 inch diameter or larger are transversely and laterally braced.

T F **FALLING DEBRIS:** Falling debris cannot damage yard and plant piping.

T F **CHLORINE CYLINDERS:** Chlorine cylinders are individually restrained.

T F **HORIZONTAL TANKS:** Horizontal tanks (including chlorine, liquid natural gas, propane, diesel, chemical) are adequately anchored.

T F **ELEVATED TANKS**: Elevated tank and equipment legs are adequately braced.

Pipelines

T F **BACKFILL AND BEDDING:** Pipes are buried in compacted bedding and fill.

T F **COUPLINGS**: Couplings are flexible with rubber gaskets.

T F **MATERIALS**: Pipes are not cast iron or asbestos cement

T F **FAULT CROSSING**: Pipes do not cross active earthquake fault zones.

T F **ELEVATED PIPES:** Elevated pipes are braced for longitudinal and transverse movements.

T F **PIPING PENETRATIONS:** Pipes have clearance and flexible couplings at wall penetrations.

T F **CORROSION:** Internal and external corrosion has been studied and does not affect seismic performance.

Storage Tanks

T F **SEISMIC SHUT-OFF:** There is an automatic earthquake-triggered shut off valve.

T F **PIPING PENETRATIONS:** Piping connections have seismic joints which allow rotation and axial movement.

T F **ANCHORAGE:** Steel tanks are anchored.

T F **ANCHOR EMBEDMENT:** Anchor bolt and strap embedment will develop yield strength.

T F **ANCHOR DUCTILITY:** Anchor bolts and straps have at least 6 inches stretch length above the foundation.

T F **WIRE WRAPPED TANKS:** Wire-wrapped concrete tank reinforcing is not corroded.

T F **SLOSHING:** Roofs and supporting columns are designed to resist the effects of sloshing water.

T F **FOUNDATIONS:** Differential settlement, liquefaction, landslides or fault rupture are not expected at this site.

T F **WELD CORROSION ALLOWANCE:** Steel tank weld thickness was increased to allow for corrosion.

T F **TANK BRACING:** Elevated tank legs are braced.

T F **COMPRESSION BRACING:** Elevated tank leg bracing has significant compression capacity.

Containment Reservoirs for Tanks

T F **LIQUEFACTION:** Earth berms will not liquefy.

T F **LINING**: The reservoir is lined.

T F **SEISMIC SHUT-OFF**: There is an automatic earthquake-triggered shut off valve.

Lifeline Support Buildings

T F **BUILDINGS:** Buildings have been evaluated and found acceptable according to FEMA procedures or P355 and P355.1.

T F **EXITS**: Suspended equipment over exit corridors is has adequate lateral bracing and vertical support.

T F **EXHAUST FANS**: Failure of exhaust fans will not create areas with a hazardous atmosphere.

T F **ANCHORAGE**: Office and lab equipment is adequately anchored.

T F **COMPUTER FLOORS:** Computer floor pedestals are braced along every grid line. Pedestals and braces are bolted to the floor.

Electrical Power

T F **OFF-SITE POWER:** Failure of off-site electrical power will not affect operations.

T F **ANCHORAGE:** Transformers, control cabinets, switchgear, motor control centers, etc. are adequately anchored.

T F **POLE MOUNTED TRANSFORMERS:** Pole mounted transformers are laterally braced and anchored.

Battery Racks

T F **ANCHORAGE: Battery** racks are anchored to the floor.

T F **BRACING: Battery** racks have diagonal bracing.

T F **RESTRAINTS:** Racks have side and end restraints located above the battery mid-height

T F **FOAM SPACERS:** Batteries have spacers between each cell and between cells and the side and end restraints.

T F **CABLES:** Cables have adequate slack, connections are tight.

Uninterrupted Power Supply

T F **ANCHORAGE:** Charger and invertor units are anchored.

Emergency Power Engine Generators

T F **ANCHORAGE:** Generator is bolted to the floor.

T F **VIBRATION ISOLATORS:** Isolators and retainers are not cast iron.

T F **SNUBBERS:** Vibration isolators have seismic snubbers.

T F **SUPPLY LINES:** Fuel, electric, cooling water, air start, exhaust and water lines can accommodate relative movement.

T F **FUEL TANKS:** Fuel tanks are adequately braced and anchored.

T F **COMMON FOUNDATIONS:** Engines and generators are on common foundations.

T F **DIESEL FUEL:** Diesel fuel is changed at least once per year to prevent clogged fuel filters and injectors.

T F **COOLING SYSTEM**: Cooling system does not leak and has enough make-up water.

T F **SYSTEM LOADS**: System loads have not increased since the generator was installed.

T F **AIR START:** Air start system compressor and air tanks are adequately anchored.

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Table 1

RELATIVE PIPELINE VULNERABILITIES

Pipe Type	Ground Shaking (MMI)		Liquefaction, Lurching	Landsliding	Fault Rupture Displacement, (Inches)	
	8	10			10	100
WSAWJ (Modern)	L	L	L	L	L	L
WSAWJ (Pre-WWII)	L	L	M	M	M	M
WSGWJ (Pre-WWII)	M	H	H	H	H	H
WSCJ	M	M	H	H	H	H
CI	M	M	H	H	H	H
AC	M	M	H	H	H	H

Source: Eguchi (1983)

Pipe Types:

WSAWJ = Welded Steel Pipe with Arc-Welded Joints
WSGWJ = Welded Steel Pipe with Gas-Welded Joints
WSCJ = Welded Steel Pipe with Caulked Joints
CI = Cast Iron
AC = Asbestos Cement

Vulnerability Levels:

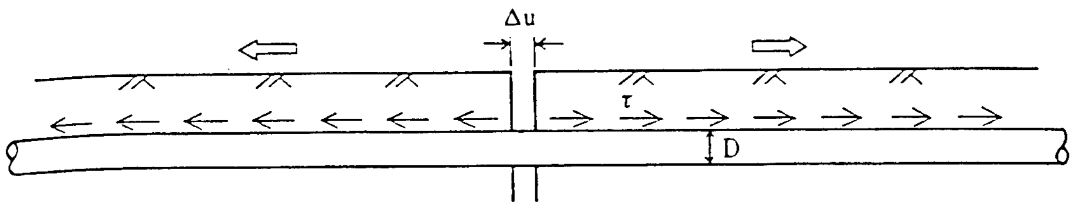
Low = No pipeline leaks expected.
Moderate = Some repairs expected.
High = Likely to fail or require repair given this level of seismic intensity or effect.

Table 2
UBC Values of C_p

NONSTRUCTURAL COMPONENTS AND EQUIPMENT	Cp
1.Elements of structures	
I. Walls including the following:	
a. Unbraced (cantilevered) parapets	2.00
b. Other exterior walls above the ground floor	0.75
c. All interior bearing and nonbearing walls and partitions	0.75
d. Masonry or concrete fences over 6 feet (1829 mm) high	0.75
2. Penthouse (except when framed by an extension of the structural frame)	0.75
3. Connections for prefabricated structural elements other than 'walls, with force applied at center of gravity	0.75
4. Diaphragms	---
2. Nonstructural components	
1 . Exterior and interior ornamentation and appendages	2.00
2. Chimneys, stacks, trussed towers and tanks on legs:	
a. Supported on or projecting as an unbraced cantilever above the roof more than one half their total height	0.75
b. All others, including those supported below the roof with unbraced projection above the roof less than one half its height, or braced or guyed to the structural frame at or above their centers of mass	2.00
3. Signs and billboards	2.00
4. Storage racks (include contents)	0.75
5. Anchorage for permanent floor-supported cabinets and book stacks more than 5 feet (1524 mm) in height (include contents)	0.75
6. Anchorage for suspended ceilings and light fixtures	0.75
7. Access floor systems	0.75
3. Equipment	
1. Tanks and vessels (include contents), including support Systems and anchorage	0.75
2. Electrical, mechanical and plumbing equipment and associated conduit, ductwork and piping, and machinery	0.75

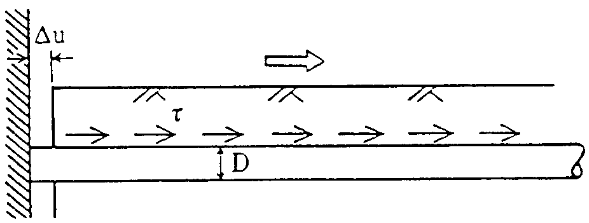
Table 3
R_w Factors For Nonbuilding Structures

Vessels, including tanks and pressurized spheres, on braced or unbraced legs.	3
Cast-in-place concrete silos and chimneys having walls continuous to the foundation.	5
Distributed mass cantilever structures such as stacks, chimneys, silos and skirt-supported vertical vessels.	4
Trussed towers (freestanding or guyed), guyed stacks and chimneys.	4
Inverted pendulum-type structures.	3
Cooling towers.	5
Bins and hoppers on braced or unbraced legs.	4
Storage racks.	5
Signs and billboards.	5
All other self-supporting structures not otherwise covered.	4

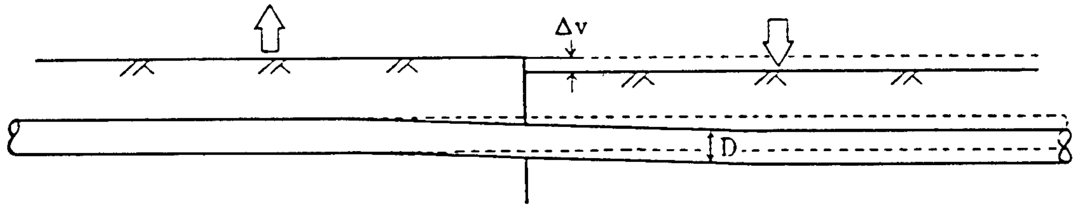


Δu : Allowable ground displacement = Deformability Index (Horizontal)
τ : Soil Restriction

a. Horizontal Relative Displacement between two Ground Blocks

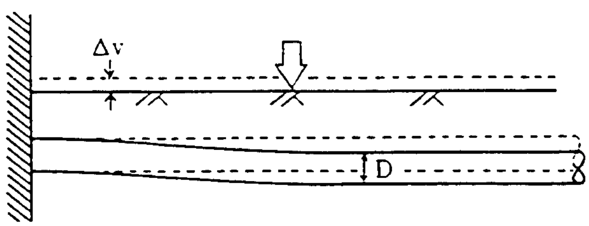


b. Horizontal Displacement in a Ground Block Adjacent to a Rigid Structure



Δv : Allowable ground displacement = Deformability Index (Vertical)

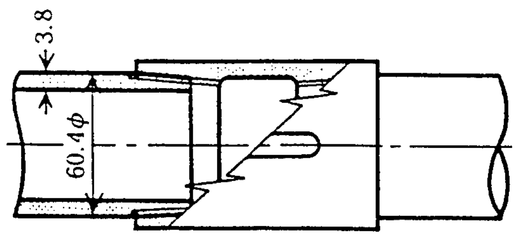
c. Vertical Relative Displacement between two Ground Blocks

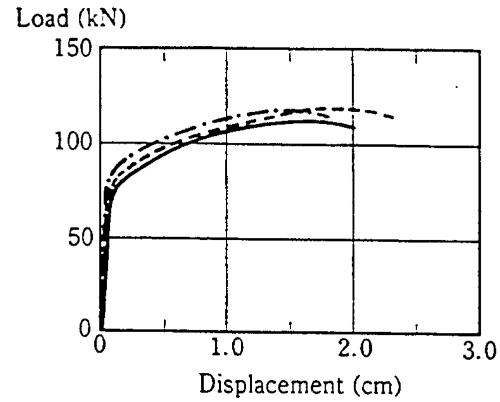


d. Vertical Displacement in a Ground Block Adjacent to a Rigid Structure

Ground-Displacement Models for Deformabilitty Evaluation for Buried Pipelines

Figure 1. Ground-displacement models for pipe deformation.

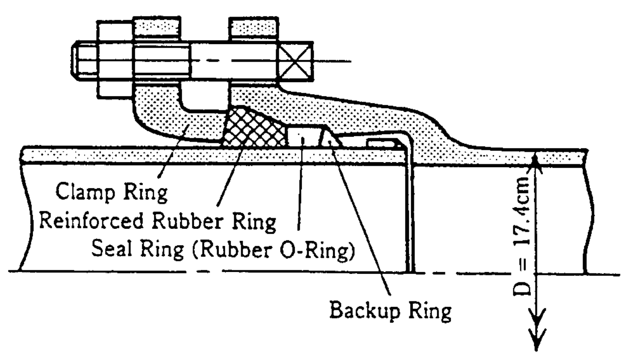


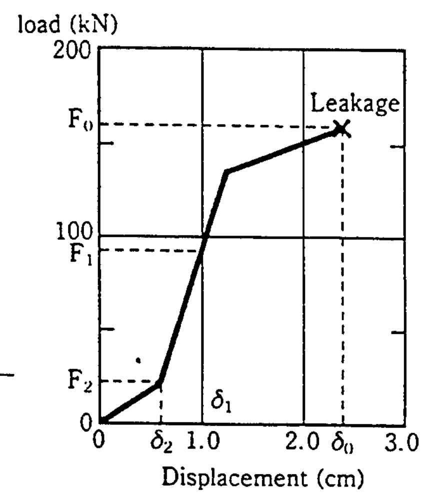


a. Cross Section of Thread-Joint (Socket, 50 mm)

b. Load-Displacement Diagram of 50 mm Thread-Joint (Axial Tension)

Cross Section and Load-Displacement Diagram of 50 mm Thread Joint



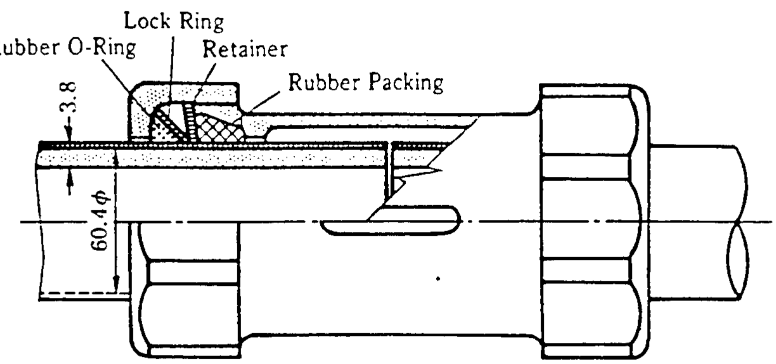


a. Cross Section of TM Joint

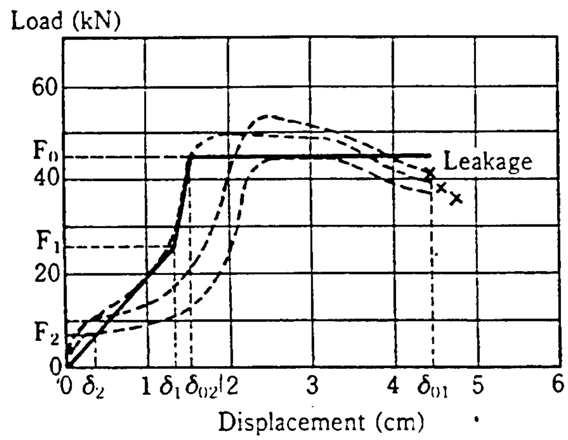
b. Load-Displacement Diagram for 150 mm TM Joint

Cross Section and Load-Displacement Diagram of Tokyo-Gas Type Mechanical Joint (TM Joint) for Ductile Cast-Iron Pipe (150 mm)

Figure 2. Cross section of Japanese pipe joints and strength. from Nishio (1992)



a. Cross Section of Mechanical Joint for Steel Pipe (Tokyo Gas LM-Type Socket, 50 mm)



b. Load-Displacement Diagram of 50 mm Mechanical Socket
----- Experiments
——— Diagram for Calculation

Cross Section and Load-Displacement Diagram of Mechanical Joint for Steel Pipe (50 mm Socket)

Figure 3. Cross section of Japanese pipe joint and strength. from Nishio (1992)

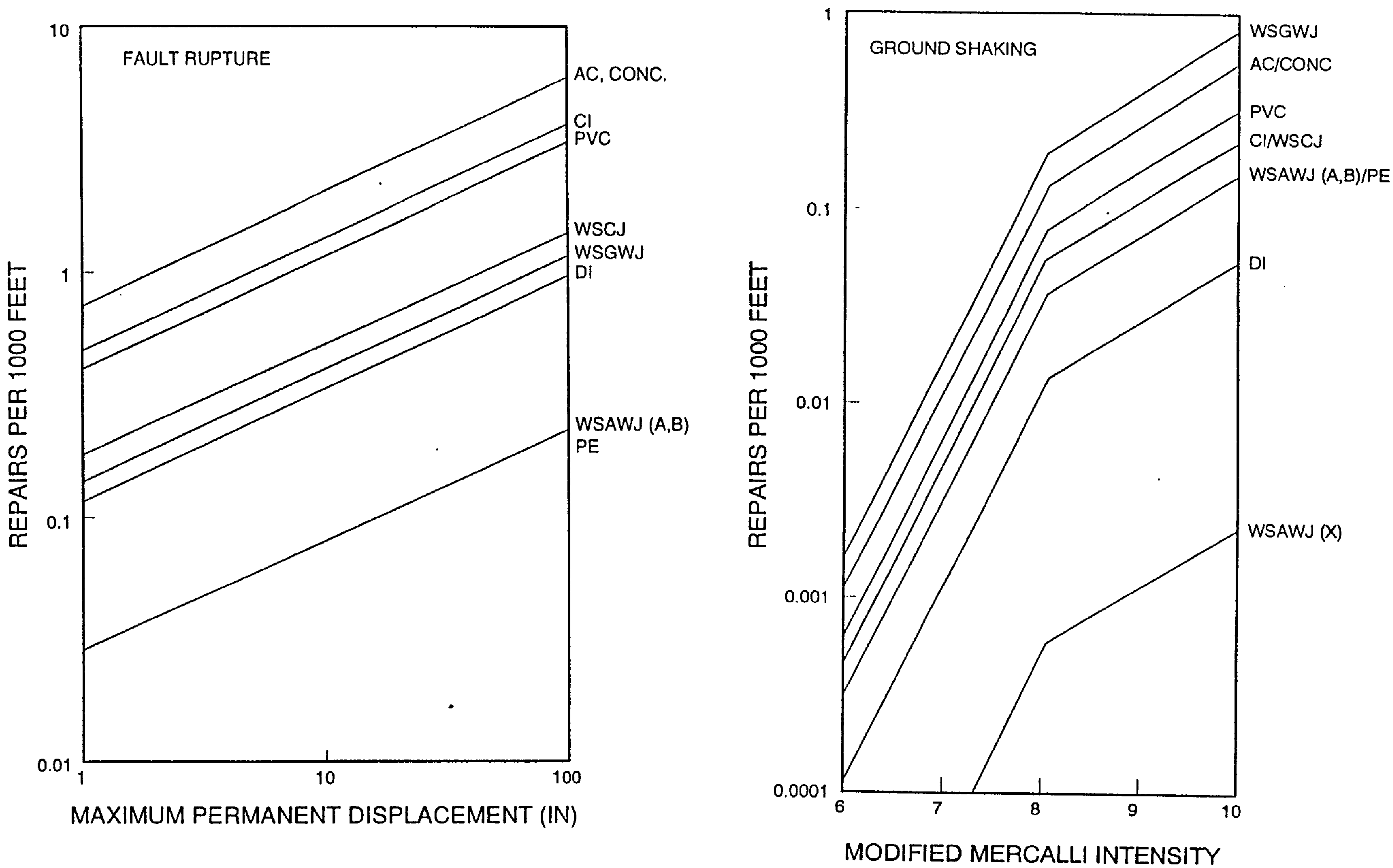
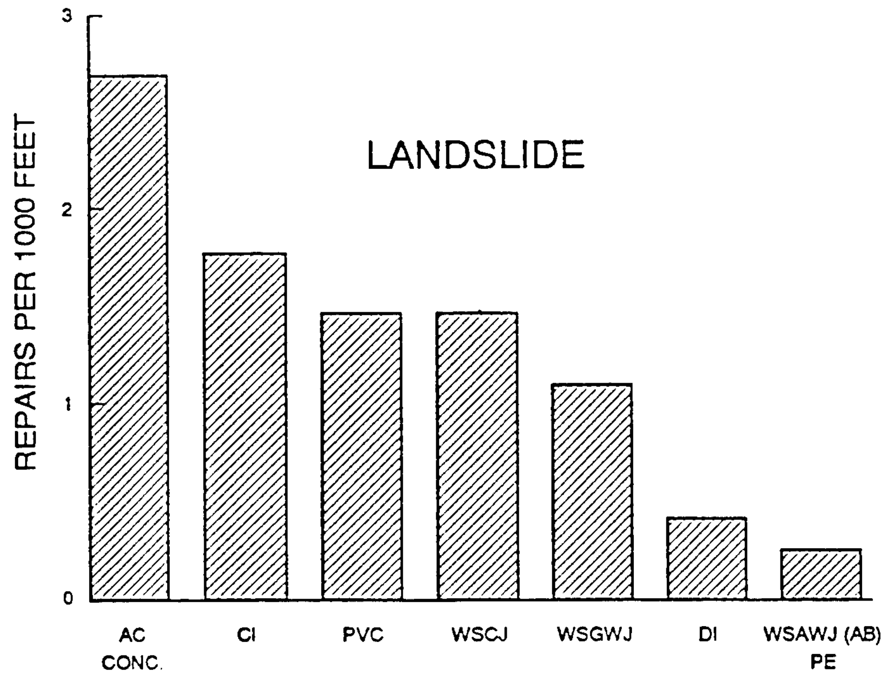


Figure 4. Pipe repair model for fault rupture and shaking, from Eguchi et al. (1994).



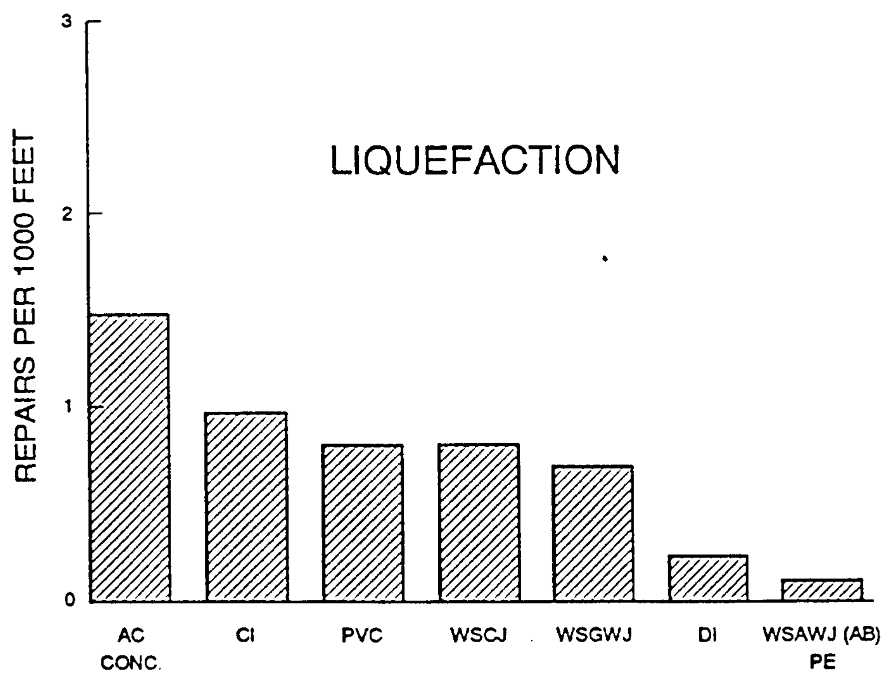
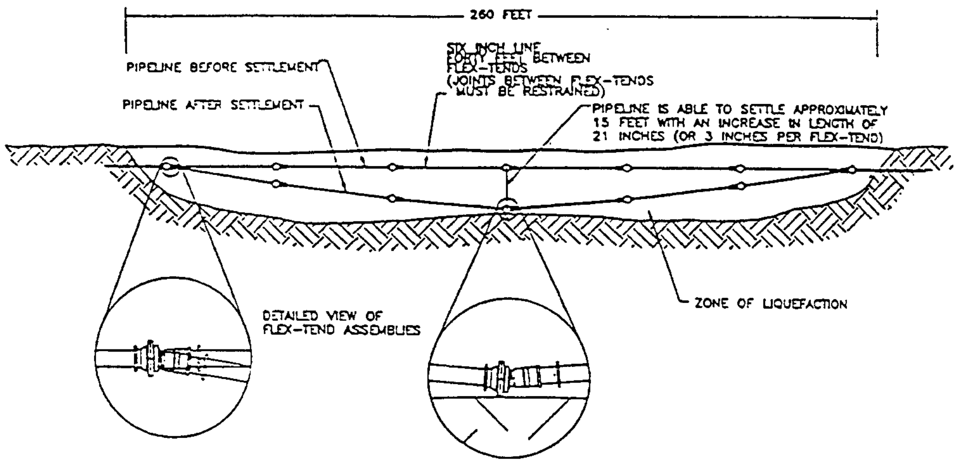
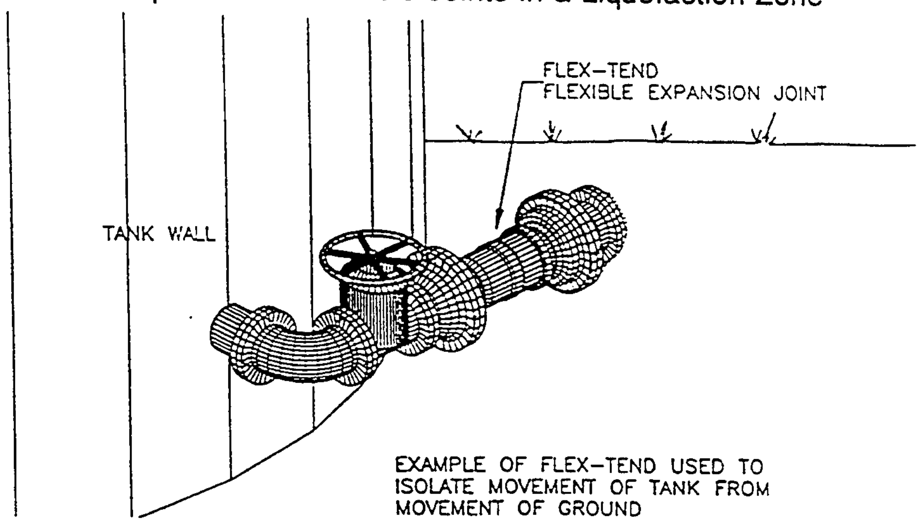


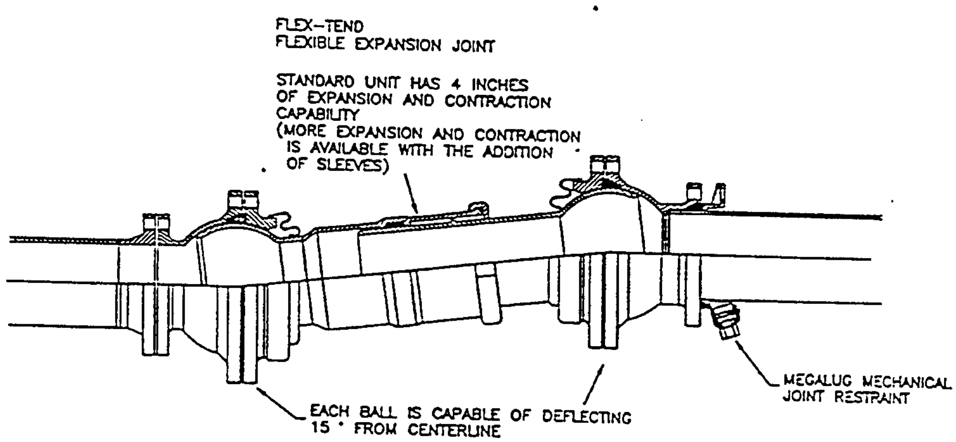
Figure 5. Pipe repair model for landslide and liquefaction, from Eguchi et al. (1994).



Pipeline with Flexible Joints in a Liquefaction Zone

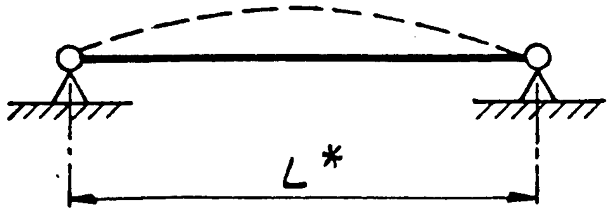


Flexible Joint for Pipe-Structure Connection



A Flexible Joint Cross Section

Figure 6. Flexible pipe connections. from Wand et al. (1992)



DIAMETER INCHES	STD.WT. STEEL PIPE 40 S	EX.STRONG STEEL PIPE 80 S	COPPER TUBE TYPE K	COPPER TUBE TYPE L	COPPER TUBE TYPE M	85 RED BRASS & SPS COPPER PIPE
1	6'-6"	6'-6"	5'-0"	4'-9"	4'-6"	5'-6"
1½	7'-6"	7'-9"	5'-9"	5'-6"	5'-6"	6'-6"
2	8'-6"	8'-6"	6'-6"	6'-6"	6'-3"	7'-0"
2½	9'-3"	9'-6"	7'-3"	7'-0"	7'-0"	8'-0"
3	10'-3"	10'-6"	7'-9"	7'-6"	7'-6"	8'-9"
3½	11'-0"	11'-0"	8'-3"	8'-3"	8'-0"	9'-3"
4	11'-6"	11'-9"	9'-0"	8'-9"	8'-6"	9'-9"
5	12'-9"	13'-0"	10'-0"	9'-6"	9'-6"	10'-9"
6	13'-9"	14'-0"	10'-9"	10'-6"	10'-3"	11'-6"
8	15'-6"	16'-0"				
10	17'-0"	17'-6"				
12	18'-3"	19'-0"				

*MAXIMUM UNSUPPORTED OR UNBRACED LENGTHS (L) ARE BASED ON WATER-FILLED PIPES WITH PERIOD (T_a) EQUAL TO 0.05 SEC. WHERE

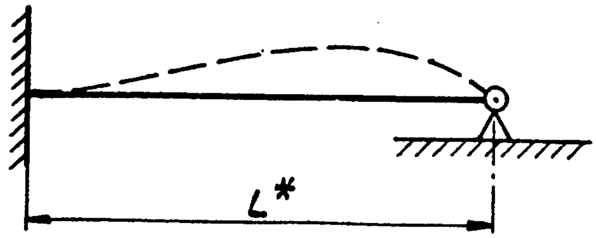
$L^2 = 0.50\pi T_a \sqrt{EIg/w}$

E = MODULUS OF ELASTICITY OF PIPE

I = MOMENT OF INERTIA OF PIPE

w = WEIGHT PER UNIT LENGTH OF PIPE AND WATER

Figure 7. Maximum Span for rigid pipe, pinned-pinned. From NAVFAC P355 Figure 12-4

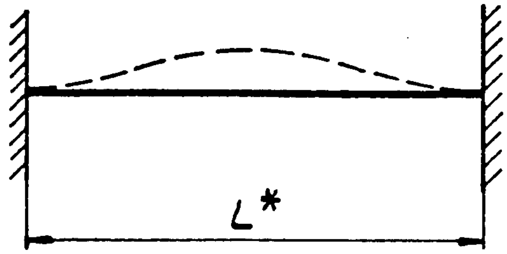


DIAMETER INCHES	STD. WT. STEEL PIPE 40 S	EX. STRONG STEEL PIPE 80 S	COPPER TUBE TYPE K	COPPER TUBE TYPE L	COPPER TUBE TYPE M	85 RED BRASS & SPS COPPER PIPE
1	8'-0"	8'-0"	6'-0"	6'-0"	5'-9"	6'-9"
1½	9'-6"	9'-6"	7'-3"	7'-0"	7'-0"	8'-0"
2	10'-6"	10'-9"	8'-0"	8'-0"	8'-9"	9'-0"
2½	11'-9"	11'-9"	9'-0"	8'-9"	8'-6"	9'-9"
3	12'-9"	13'-0"	9'-9"	9'-6"	9'-3"	10'-9"
3½	13'-6"	14'-0"	10'-6"	10'-3"	10'-0"	11'-6"
4	14'-6"	14'-9"	11'-0"	11'-0"	10'-9"	12'-3"
5	16'-0"	16'-3"	12'-3"	12'-0"	11'-9"	13'-3"
6	17'-0"	17'-9"	13'-6"	13'-0"	12'-9"	14'-3"
8	19'-3"	20'-0"				
10	21'-3"	22'-0"				
12	23'-0"	23'-6"				

*MAXIMUM UNSUPPORTED OR UNBRACED LENGTHS (L) ARE BASED ON WATER-FILLED PIPES WITH PERIOD (T_a) EQUAL TO 0.05 SEC. WHERE

$$L^2 = 0.78\pi\, T \sqrt{EIg/w}$$

Figure 8. Maximum Span for rigid pipe, fixed-pinned. From NAVFAC P355 Figure 12-5



DIAMETER INCHES	STD. WT. STEEL PIPE 40S	EX. STRONG STEEL PIPE 80S	COPPER TUBE TYPE K	COPPER TUBE TYPE L	COPPER TUBE TYPE M	85 RED BRASS ¢ SPS COPPER PIPE
1	9'-6"	9'-6"	7'-3"	7'-3"	7'-0"	8'-0"
1½	11'-6"	11'-6"	8'-6"	8'-6"	8'-3"	9'-9"
2	12'-9"	13'-0"	9'-9"	9'-6"	9'-6"	10'-9"
2½	14'-0"	14'-3"	10'-9"	10'-6"	10'-6"	11'-9"
3	15'-6"	15'-9"	11'-9"	11'-6"	11'-3"	13'-0"
3½	16'-6"	16'-9"	12'-6"	12'-3"	12'-0"	14'-0"
4	17'-3"	17'-9"	13'-6"	13'-0"	13'-0"	14'-9"
5	19'-0"	19'-6"	15'-0"	14'-6"	14'-3"	16'-0"
6	20'-9"	21'-3"	16'-3"	15'-9"	15'-6"	17'-3"
8	23'-3"	24'-3"				
10	25'-9"	26'-6"				
12	27'-6"	28'-6"				

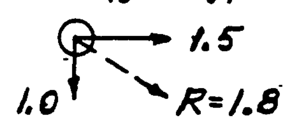
* MAXIMUM UNSUPPORTED OR UNBRACED LENGTHS (L) ARE BASED ON WATER-FILLED PIPES WITH PERIOD (T_a) EQUAL TO 0.05 SEC. WHERE

$$L^2 = 1.125\pi T_a \sqrt{EI g/\omega}$$

Figure 9. Maximum Span for rigid pipe, fixed-fixed. From NAVFAC P355 Figure 12-6

Diameter (in.)	Std. Wgt. Steel Pipe - 40S		Ex. Strong Steel Pipe - 80S		Copper Tube Type L	
	L*(ft)	F†(lbs)	L*(ft)	F†(lbs)	L*(ft)	F†(lbs)
1	22	70	22	80	11	17
1-1/2	25	140	26	180	12	35
2	29	220	30	290	14	70
2-1/2	32	380	33	460	15	110
3	34	550	35	710	17	150
3-1/2	36	730	38	930	18	220
4	39	960	40	1,200	19	300
5	41	1,440	44	1,900	20	470
6	45	2,120	46	2,750	22	730
8	49	3,740	54	5,150	26	1,550
10	54	6,080	59	7,670	28	2,620
12	58	8,560	61	10,350	31	3,950

*Maximum spans (L) between lateral supports of flexible piping are based on the resultant of an assumed loading of 1.5 w ($ZI_pA_pC_p$ = 1.5) in the horizontal direction and 1.0 w (gravity) in the vertical direction. The resultant is 1.8 w.



The assumed maximum stress is 20,000 p.s.i. for steel and 7,000 p.s.i. for copper. Simple spans (pinned-pinned) are assumed. The calculated maximum lateral displacements are 3.5 inches for steel (E = 29 x 10^6 p.s.i.) and 0.6 inch for copper (E = 15 x 10^6 p.s.i.).

†The horizontal force (F) on the brace is based on 1.5 w L for the maximum span. For shorter spans, $F_{design} = (L_{design}/L)F$.

Figure 12-7. Maximum span for flexible pipes in Seismic Zone 4.

Zone	L (feet)	F (pounds)	$ZI_pA_pC_p$
3	1.1	0.8	1.12
2B	1.20	0.6	0.75
2A	1.25	0.5	0.56
1	1.35	0.3	0.28

Table 12-2. Multiplication factors for figure 12-7 for Seismic Zones 1, 2, and 3 or for cases where $ZI_pA_pC_p$ is not equal to 1.5.

Figure 10. NAVFAC P355 Figure 12-7 and Table 12-2.

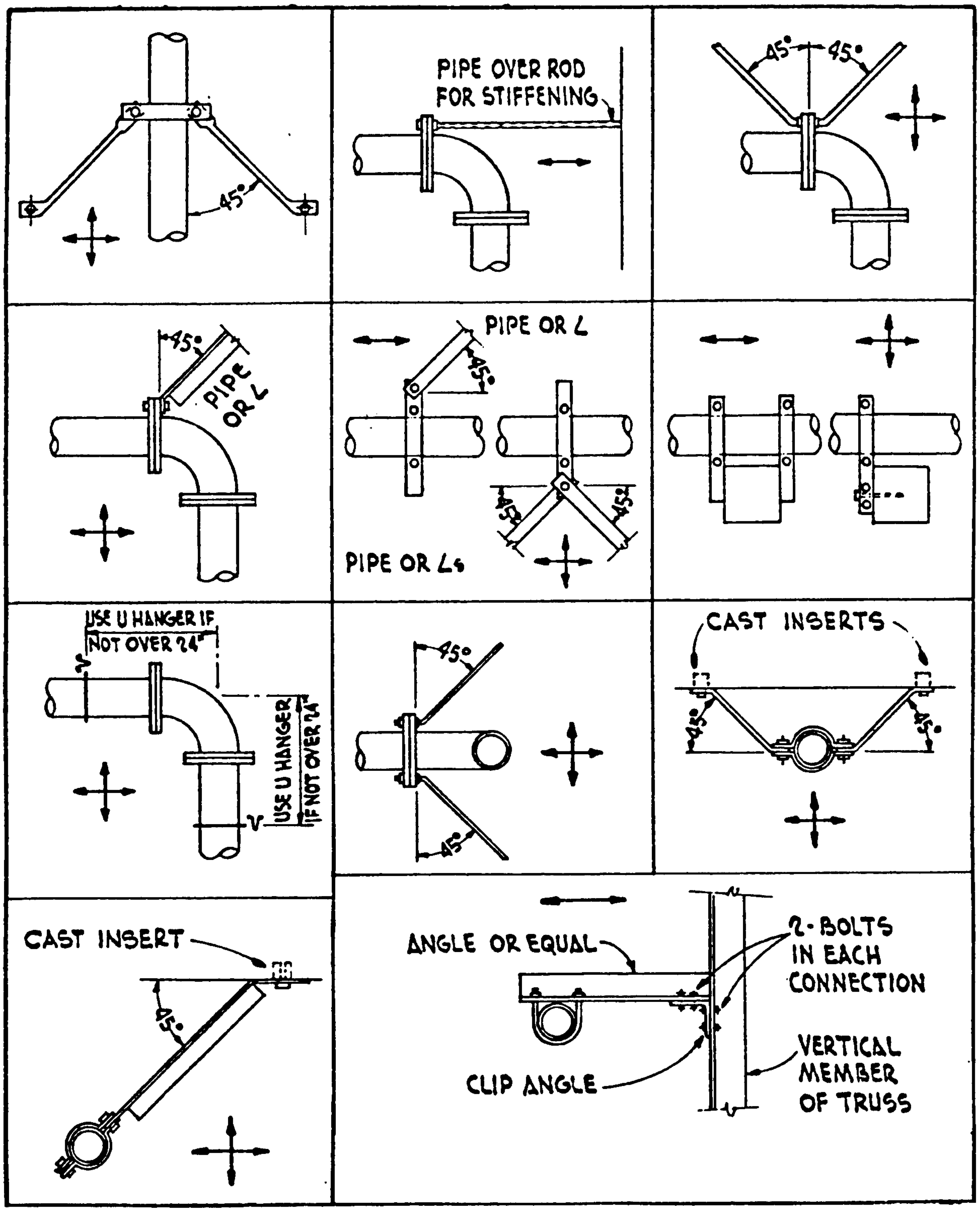


Figure 11. Acceptable seismic details for sway bracing from NAVFAC P355 Figure 12-8.

Example

Slack Requirements of Linetrap

Pole Length	L = 15 feet x 12 = 170 inches
Material Steel	E = 29,000,000 psi
Pole Moment of Inertia	$I = 65.6 \text{ in}^4$

Weight of Linetrap + half pole. W= 850 lbs

$$K = \frac{3EI}{L^3} = 1161.7 \text{ lb/in}$$

Computer solution using Pacoima record:

```
SINGLE DEGREE HYSTERETIC MODEL
AVERAGE OF MAXIMUM ACCELERATION =  0.180
RATIO FIRST YIELD POINT   S/W   =  1.176
RATIO OF K2 TO K1 (REDUCTION)   =  1.000
DAMPING RATIO                   =  0.020

INITIAL NAT. PERIOD    =    0.273
TIME INCREMENT         =    0.008
1ST YP   DISPLACEMENT  =    0.856  IN

MAXIMUM DISPLACEMENT  =      0.335  IN

EFF NATURAL PERIOD     =    0.273
MAX HORIZ FORCE  F     =      391.36
```

DISPLACEMENT RESPONSE OF OSCILATOR IN INCHES

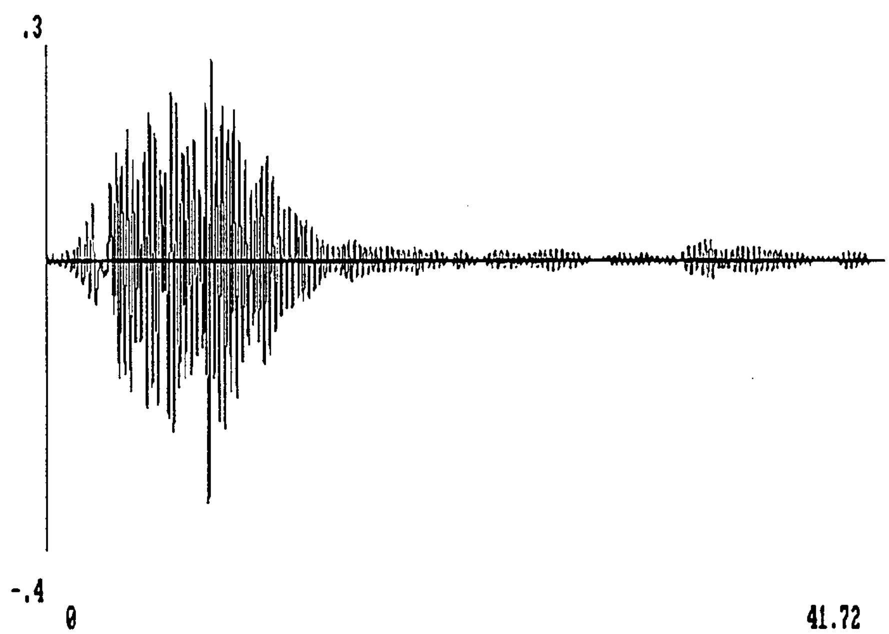


Figure 12 Sample calculation of slack deflection.