AmericanLifelinesAlliance

A public-private partnership to reduce risk to utility and transportation systems from natural hazards and manmade threats

Seismic Guidelines for Water Pipelines

March 2005

FEMA

National Institute of BUILDING SCIENCES

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Seismic Guidelines for Water Pipelines

Prepared for:
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As part of the:
American Lifelines Alliance

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C1.0 Commentary

These Guidelines are intended to be consistent with existing codes and expected community response. The UBC, IBC, and other codes have already established precedent on how the community is intended to respond following an earthquake. It is proper for the water pipelines to be designed consistent with the rest of the community and therefore assumed that the importance of certain facilities described in existing codes requires that not only the facility perform to a minimum level to protect human life during and following an earthquake, but that the water system perform adequately so that in an event of a disaster rescue crews will have adequate fire and potable water supplies to perform adequate emergency response activities. There is an assumed one to one relationship between the code facility design and the recommended performance of the pipeline service the facility. There is a certain level of risk for different facilities already accepted by the community based on existing codes and water pipelines need not exceed this risk acceptance level. At the same time water pipelines generally serve a greater portion of the community than a typical building facility. As a result, there are limitations set on pipeline design to ensure that community fire and potable water service following an earthquake is met without regard to the type of facility the pipelines are providing service. In general, the more important the facility, the more stringent the seismic design requirements.

C1.1 Objective of the Guidelines

When we use the term "cost effective", what we mean is that the incremental cost to install a pipeline with seismic-resistant features should not be so large such that the net present value of the benefits is less than the incremental cost. The "benefits" are the reduction is losses from future earthquakes, discounted to the present time. These benefits include the reduction in repair costs form avoided pipe damage; reduction in economic impacts to society should water not be delivered for a time after an earthquake, including impacts from fires and unavailability of water for residential and commercial purposes; and other impacts that might be site-specific, such as reduction in inundation losses, reduction in erosion losses, and (rarely) reduction in life safety impacts.

For example, the end user might design every 6-inch diameter pipe in a water distribution system to the requirements for Function Class IV pipes, where few if any pipes will break in rare earthquakes. This would result in a very reliable water system. However, based on the experience of the authors of the Guidelines, the extra expense would likely not payoff in the long run, and a less expensive solution, relying on emergency response capability with some limited pipe damage, is likely more cost-effective. However, nothing in these Guidelines should limit the owner from installing a higher Function Class pipeline than would otherwise be recommended fro a strictly cost-effectiveness test, albeit with the recognition that the rate payer will have to shoulder this burden.

C1.2 Project Scope

There will sometimes be cases where the Chart Method is too general, or the pipe too important. In these cases, the designer can use either the ESM or FEM methods to refine and further quantify the design. In some situations, it might turn out that the three methods will result in different designs. As Guidelines, we make no statement that a particular design is "safe" or "unsafe", and we recognize that there may be inconsistencies between the three methods. Generally speaking, the FEM method will provide the most quantified information that can be used in design. If a situation arises that the designer finds that there is conflict between the three methods, then the design should revert to basic principles as to what he is trying to achieve: namely, an overall water system performance after rare earthquakes which does not overly impact the community. These Guidelines specifically allow that some pipe damage is acceptable to the community, so a modest over-design or under-design at any one location might not be overly important when taking the larger view of the community as a whole.

C1.4 Limitations

While every effort has been made to develop a set of Guidelines that are clear, concise and comprehensive, the authors feel that we have not accomplished these goals entirely. There are technical areas throughout the document that are, as of 2005, still not entirely agreed upon in the engineering community.

With time, the engineering community will have better geotechnical models to describe the hazards; better analytical techniques to evaluate the forces and displacements imposed on pipelines; better understanding as to the nature of corrosion and other time-varying effects on pipes; and new pipe products made available by pipe vendors. In all these cases, we endorse the efforts by the end-user to use techniques that may not be included in these Guidelines, as long as these techniques are consistent with achieving a cost effective water pipe network.

It is left to a future committee(s) to evaluate these Guidelines before adoption into codes and standards. This can be done in several steps. First, a series of trial designs can be developed using the Guidelines, including cost estimates. Second, the Guidelines should be updated to reflect the observations from the trial designs. Third, pipe manufacturers should be encouraged to develop catalog information that is needed to apply some of the procedures in the Guidelines. Fourth, code and standard setting organizations can adopt the Guidelines as may be suitable.

The authors hope that these Guidelines will help overcome the ongoing vulnerability facing our existing water pipeline infrastructure. It is our hope that with time, the vulnerabilities will be reduced, and there will be no repeats of the widespread collapse of water systems with ensuing fire losses in the 1906 San Francisco, 1923 Kanto (Tokyo), 1995 Hanshin (Kobe) and other historical earthquakes.

C2.0 Project Background

Three methods of analysis and design are provided in these Guidelines. Each method has its pros and cons. The authors of the Guidelines decided that the three methods are suitable, in order to provide procedures that are:

- Simple and Each to use (Chart Method)
- Follow fundamental strength of material concepts (ESM)
- Comprehensive (FEM)

We expect that perhaps 75% to 90% of all pipe installations can be reasonably designed using the Chart Method. Only the most important pipelines will usually be designed using either the ESM or FEM. It is understood that there are intrinsic differences between the three methods, and the three methods may result in different design solutions. Recognizing that the fundamental goal of the Guidelines is to develop a water pipeline network that will not suffer too much damage in rare earthquakes, we are not concerned that the three methods may result in different designs for specific pipelines, as long as that fundamental goal is achieved.

C2.2 Hydrodynamic Loading

There is increasing evidence that earthquake-induced hydrodynamic loading plays a role in pipe damage to segmented pipes. As the earth vibrates, and the pipe with it, the water is alternately accelerated in each direction at elbows and bends in the pipe. This causes traveling waves within the pipe.

Equation C8-1 of the commentary provides a simple formula that can be used to estimate the extra load on pipes at elbows and bends due to seismic-induced hydrodynamic loads. Conceptually, the rapid valve closure formula in Section 6 could be used, replacing the change in velocity of the steady state water flow to the change in velocity imparted by the ground motion at a bend; although this approach neglects the transient nature of the ground motions.

Equation C8-1 may not apply to all pipeline configurations, and there are currently no nomographs available to simply apply these findings to all pipe configurations and seismic hazards. In the Chart Method, the Guidelines recommend that all segmented pipe have three restrained joints adjacent to each bend, in areas with high seismic hazard; and this will likely materially reduce the damage rate for such instances. In the FEM method, the user could perform an analysis similar to the sort described in Section C8.0, to quantify the loadings for the particular situation.

C2.3 Guidelines Context

These Guidelines were complete in early 2005. Several of the authors of these Guidelines attended a joint JWWA-AWWARF workshop, held in late January 2005 in Kobe Japan, hosted by the City of Kobe Water Utility, to coincide with the ten-year anniversary of the Great Hanshin (Kobe) earthquake of January 17, 1995. This workshop was a gathering of 60 engineers from more than 25 Japanese and American water utilities, as well as academic researchers from Japanese, American and Taiwanese universities and institutions. The workshop was dedicated to an understanding of what happens to water utilities in earthquakes, and what can be done to mitigate the impacts. This was the fourth such workshop, the prior being held in Oakland (1999, host East Bay Municipal Utility District), Tokyo (2001 host City of Tokyo Water Utility), Los Angeles (2003, host Los Angeles Department of Water and Power).

During the course of these workshops, many aspects of seismic impacts to water utilities have been shared. By the third workshop in Los Angeles, a theme was apparent. In a simple form, the theme is as follows:

- Japanese water utilities are in the process of replacing a considerable portion of their pipeline inventory. Their intent is to replace older, vulnerable water pipelines with newer *seismically resistant* pipelines. In 1990, perhaps less than 1% of all Japanese water pipelines were then of the seismically resistant type. By 2004, about 15% of all Japanese pipelines had been replaced with seismically resistant pipelines. As of 2004, there is on ongoing Japan-wide rate of older water pipe replacement of about 10,000 km per year. In 2004, perhaps 10% of the entire capital investment made by Japanese water utilities was devoted to replacement of highly vulnerable pipelines with newer seismically-resistant pipelines.

- In contrast, American water utilities in high seismic regions (like coastal California, Seattle, Portland, Memphis and St. Louis) are not replacing their vulnerable pipelines with newer seismically resistant pipelines at anywhere near as rapid a rate as being done in Japan. Estimates of pipe replacement by the large water utilities such as Los Angeles, San Francisco, San Diego, EBMUD, Portland and Seattle range around 0.2% of inventory per year; replacement pipes are not always seismically resistant.

After the 2003 workshop, engineers from several of the American water utilities, augmented by leading US academic researchers and consulting engineers, got together to form a working group funded by the ALA to examine the apparent discrepancy in strategy between the Japanese and American water utilities. This group of engineers developed these Guidelines.

At the 4th workshop in Kobe in January 2005, a draft version of these Guidelines was presented to a panel of about 45 Japanese water utility engineers. Over the course of several days, formal and informal conversations and comments were held between the American and Japanese delegates. A few of these are summarized below:

- Japanese water utilities (including Kobe, Osaka, Hiroshima, Tokyo and many others) are actively replacing old cast iron pipe with brittle joints (and in some cases thin walled small diameter screwed steel pipe, asbestos cement pipe, and ductile iron pipe with push-on joints) with newer "seismic resistant" pipes. By "seismic resistant" pipe, the Japanese refer to ductile iron pipe with SII (chained) type joints, and larger diameter steel pipe with welded joints. The pipes earmarked for early replacement include those pipes traversing though liquefiable soils and also those pipes of larger diameter serving larger populations. Seismic mitigation programs being planned and implemented in Japan range up to $5 billion (for the largest utilities), with implementations to be done over tens of years. These seismic mitigation programs cover pipe replacement, as well as adding redundancy, seismic upgrade of older tanks, seismic improvement to dams, improvement in post-earthquake disaster planning and recovery strategies, including GIS-based systems to map damage and restoration efforts, community outreach, and other factors.

- American utilities (including EBMUD, San Francisco, Los Angeles, CCWD, Seattle, Portland) are doing many similar activities as their Japanese counterparts, including actively upgrading tanks, hardening water treatment plants, improving dams, adding redundancy, improving emergency response.

- The major difference between Japanese and American seismic mitigation programs is that the Japanese include pipe replacement as a major element of their mitigation strategy, whereas Americans do not.

In preparing these Guidelines, the authors have asked themselves: Are the Japanese right in pursuing substantial pipe replacement? Are the Americans right in not actively pursuing much pipe replacement? Are both right? Are both wrong?

Factors that suggest that the recent Japanese practices (high rate of pipe replacement) are right include the following:

- The 1995 Great Hanshin earthquake resulted in 1,757 pipe repairs to be made just within the City of Kobe water system (there were many more water pipes to be repaired in neighboring cities, as well as tens of thousands of damaged service line laterals that are not counted above). The main office building of the Kobe water utility collapsed; and this hampered orderly response to managing the restoration effort. It took 10 weeks for water to be essentially completely restored to customers via the pipe network. At the time of the earthquake, about 5% of the pipe inventory in Kobe were "seismic resistant" pipelines, and these apparently suffered no damage, even when exposed to PGDs of inches to sometimes a foot or more. Other pipelines (cast iron, ductile iron with push-on joints) suffered a lot of damage. The loss of water supply in Kobe caused great economic and social harm. Fires Following Earthquake attributed to about 10% (559 people) of all mortalities (about 6,000 people).

- The 1923 Great Kanto (Tokyo) earthquake resulted in widespread water pipeline damage. Essentially all pipes at that time would be classified as non-seismic resistant. The subsequent fires led to arguably over 100,000 casualties over-and-above that cause by damage to structures due to the ground shaking alone.

- Japanese cities have generally higher densities than US cities, meaning that one mile of water pipeline serves a greater number of people in Japan than in the US. Thus, pipe replacement of a length of Japanese pipe might have greater benefit than in the United States.

- Many areas of Japan have higher seismicity than US cities. This means that potentially damaging earthquakes occur more often in Japan than in the United States. This increases the sensitivity of Japanese to the need for seismic-resistant pipelines. Analytically, this also means that there is a higher benefit cost ratio for Japanese pipe replacement to US pipe replacement, all other factors being equal.

Factors that suggest that recent US practices might be right (low rate of pipe replacement) include:

- Recent large earthquakes in the United States, including the 1989 Loma Prieta and 1994 Northridge, did not result in long term water outages to significant populations. Fires that ignited in these earthquakes were largely controlled, and only a few hundred structures burned and there were no fire-related casualties.

- US utilities are loathe to increase water rates to fund major pipe replacement efforts.

- US utility managers and public directors are of the opinion "if it ain't broke, don't fix it"

- US utility managers might be of the opinion that it is easier to "manage the damage" than mitigate before the earthquake.

Factors that suggest US practices might be wrong include:

- Until the development of these Guidelines, there have been no industry-wide seismic requirements for water pipelines. This has led to ongoing pipe installation practices which might be good enough to hold water and not leak too often under normal (non-seismic) loads. While the style of construction of existing pipelines varies between water utilities, it is reasonable to say that at least one-third to as much as three-quarters of all pipes installed in San Diego, Los Angeles, San

Francisco, EBMUD, CCWD, Portland and Seattle are still highly vulnerably to major damage if subjected to PGDs of a few inches or more.

- The institutional memory of the damage to San Francisco (and smaller nearby cities) from the 1906 earthquake is largely gone in the minds of current-day water utility managers. The 1906 earthquake resulted in 300 distribution pipe failures just in San Francisco (out of 400 miles of installed cast iron pipe), plus more than 30 breaks in the large diameter transmission pipes that brought water to San Francisco. The loss of water supply contributed to a great fire conflagration and the largest fire loss (as measured in current dollars) in US history. In the ensuing decades, it has been sometimes remarked by fire-loss underwriters that it is bewildering that San Francisco has not since burned down again.

- The limited fire loss damage in the 1989 Loma Prieta earthquake can be at least partially (or possibly completely) explained by the total absence of wind at the time of that earthquake. If it has been blowing at 10 mph (average speed) at the time of the earthquake, the ignition in the Marina district (where due to PGDs, there were many water pipe breaks and there was no water pressure from either the main water system, nor the backup water system) would have spread, most likely resulting in a major conflagration. The same fortunate weather condition (almost no wind) was in place at the time of the 1994 Northridge earthquake. Between these two earthquakes, there were more than 130 fire ignitions, but less than 300 burned structures. Some would say: "we were lucky".

Unlike Japan, the American pipeline manufacturers do not currently (early 2005) offer for sale a low-cost "chained" ductile iron pipe. In Japan, the Kubota Company offers such a pipe, using a "SII" joint capable of limited extension and rotation before locking up; Section 10 of these Guidelines describes this joint. Lacking the availability of a commonly available and not-too-costly pipe product, American water utilities buy what is available. Today (early 2005) more than 95% of all distribution pipeline installations use PVC pipe with push-on joints or ductile iron pipes with push-on joints; neither of these types of pipes are considered to be "seismic resistant" when exposed to much, if any, PGDs.

We hope, by the introduction of these Guidelines, that the practice of US water pipe installations in seismic regions will change. Without question, American and Japanese all agree that push-on type joints cannot be relied upon when exposed to much PGD. The incremental cost to install a new pipe with seismic-resistant features through areas prone to PGDs is considered to be well worth the money, and on this point, there is no disagreement between Japanese and Americans. With respect to pipe replacement, it is still an open question as to whether the cost of upgrade for seismic-purposes alone is worth the initial investment, at least in high seismic American cities.

To summarize, the authors are unanimous that *all* new pipe installations be designed in accordance with the provisions of Guidelines. New pipe installations include those required for new subdivisions; uprating of fire flows or demands that cannot be supported by existing older pipes; and pipe relocations caused by urban renewal and other factors. By following this strategy, the seismic ruggedness of water systems will gradually increase, and 50 to 100 years from now the water systems will be much better than they are today.

The authors cannot now make a recommendation to apply these Guidelines as the sole reason to retrofit normal distribution pipelines. The authors suggest that water utilities should seriously consider use of these Guidelines for retrofit purposes for its most important and non-redundant transmission and sub-transmission pipelines; to varying extent, this reflects the strategy adopted or being considered by EBMUD, CCWD, Portland, Seattle, and Los Angeles water utilities, amongst others.

The authors do not advise wholesale replacement of cast iron (caulked joint); asbestos cement (push on rubber gasket joint), PVC (push on rubber gasket joint) or ductile iron (push on rubber gasket joint) distribution pipe in areas subject to strong ground shaking but without PGD. Particularly bad-acting pipe, such as thin walled small diameter screwed steel pipe (often pre-dating 1940); or any pipe that needs repair more than once every five years (per 1,000-foot length) should be carefully considered for replacement with new pipe that complies with these Guidelines.

It is the hope of the authors that these Guidelines will lead to advancement in US water utility pipeline installation practices that will greatly reduce the potential for long term outages and fire following earthquake conflagrations. To achieve this goal will require substantial capital investment, possibly taking decades to fully realize. We hope that over time, new pipe products will be made commonly available for water utilities that provide the desired seismic performance at a suitably low cost.

The specific recommendations made in these Guidelines will be subject to revision and improvement as we continue to gain more experience with seismic response of existing and newly-designed pipelines. It is our intention that these Guidelines be ultimately codified into Standards and Codes. In the interval of release of these Guidelines to adoption as mandatory code, a number of steps should be taken, including the following suggested steps:

- Trial implementation for several actual installations. This can be done by water utility engineers, consulting engineers, contractors and pipe manufacturers. This should cover typical new subdivision installations, as well as pipe rehabilitation projects. The effort should cover geographically diverse areas such as coastal California, the Puget Sound area, western Oregon, and other moderate to high seismic regions in the USA.

- Lessons will continue to be learned from future earthquakes as to the performance of various types of pipelines to ground shaking, liquefaction, landslide and fault offset. These lessons learned should be considered in application of the Guidelines.

- New pipe materials and joinery will be developed and made available to water utilities. Pipe manufacturers should list the actual strength and displacement capacities of all their products.

C3.0 Performance Objectives

A water system performance goal is determined through consideration of target performance levels in relation to a seismic hazard (e.g., 90 percent of customers restored within 3-days following an earthquake having a 10% chance of exceedance in 50-years). The ASCE Guidelines (Eidinger and Avila, 1999) provide performance goals for water utilities.

The selection of the level of earthquake hazard for vulnerability assessment of large spatially separated network systems is often done using a deterministic scenario approach. In such assessments, the owner usually needs to set some system-wide performance goals, for use as benchmarks as to whether or not his water system is "good enough" under various size earthquakes. In Eidinger and Avila (1999), a series of performance goals are listed. Since 1999, these goals have been adopted, usually with minor adjustments, by many water US utilities to reflect their particular circumstances. Overall, these goals remain a reasonable starting point for establishing what constitutes an acceptable level of post-earthquake water system performance in a cost effective manner.

A pipeline's function within the system identifies its importance in achieving the system performance goal. These Guidelines are intended to be used for pipeline components within a water system and therefore does not make any specific recommendations for system performance goals and only describes pipelines in terms of their function within a system. A pipe function identifies a performance objective of an individual pipe (e.g., certain critical pipelines serving critical facilities remain operational during and following an earthquake), but not that of an entire system. It is useful and recommended, but not necessary for use of the Guide, that system wide performance objectives be established in relation to seismic design of pipelines. Section C3.2.2.6 provides a simple way to quantify performance levels of particular water pipe networks as a function of time, and these can be used to help establish the Function Class of particular pipelines.

C3.1 Categories of Pipelines

These Guidelines define water pipelines as one of four types: transmission, sub-transmission, distribution and service lateral / hydrants. The definitions we have suggested for these four types are necessarily arbitrary, and the terminology used by various water utilities could vary from that described in these Guidelines. For example, LADWP calls its largest potable water pipelines "trunk lines"; EBMUD calls some of its largest potable water pipelines "aqueducts".

A particular water utility can apply these Guidelines for their own water system and use different terminology for the various pipeline categories. Whichever terminology is used, we suggest that the performance objectives be set consistent with the concepts presented in the Guidelines.

C3.2 Pipe Function Class

C3.2.1 Pipe Function Class

Different types of pipelines in water supply and distribution networks serve different functions. For example, aqueducts transport large quantities of raw water for treatment, trunk lines transport raw or potable water supplies from treatment facilities and large storage reservoirs to delivery points feeding mains, supply mains deliver water from supply sources or trunk lines to a distribution mains, distribution mains distribute water to individual customers, and service laterals convey water from the mains directly to the facility served. Aqueducts, trunk lines, mains, and service connections identify the pipe type. The pipe function is related to its importance in providing water supply to the community and individual facilities.

The intent of the proposed method for classifying a pipe's Function is to be consistent with providing a cost effective approach to constructing and maintaining a water pipeline network with the threat of rare but real earthquakes. It is proper for the water pipelines to be designed with a philosophy consistent with the rest of the community, *but at present time (early 2005) it is entirely up to each individual water utility to choose their own performance goals and the manner in which it thinks it most suitable and cost effective to meet them; nothing in these Guidelines should be considered mandatory*.

The importance of certain facilities described in existing building codes (like UBC, IBC) requires that not only the facility perform to a minimum level to protect human life and property during and following an earthquake, but also that the water system perform adequately so that disaster rescue crews will have adequate fire and potable water supplies to reasonably perform emergency response activities. For these Guidelines, we assume there is a relationship between the code facility design and the recommended performance of the pipeline service to the facility. There is a certain level of risk for different facilities already accepted by the community based on existing code; the authors agree that water pipelines need not exceed this risk acceptance level, and some level of damage to a water network should be acceptable after rarely occurring earthquakes.

At the same time water pipelines generally serve a greater portion of the community than a typical building facility. As a result, there are limitations set on pipeline design to ensure that community fire and potable water service following an earthquake is met without regard to the type of facility for which the pipelines are providing service. In general, the seismic design requirements become more stringent with increased importance of the facility served and the greater threat to human life and property in the event a pipe is severely damaged.

With these precepts in mind, we suggest the following guidance on how to classify pipes as Function I through IV:

Function I: Pipelines that represent a very low hazard to human life and property in the event of failure. These pipelines primarily serve for agricultural usage, certain temporary facilities, or minor storage facilities. The pipelines provide potable water supply for maximum of 50 service connections and are not needed for any level of fire suppression following a significant earthquake. A Function I pipeline could also include a raw water transmission line, should failure of that line not impact the local community, owing to the availability of suitable terminal storage (or other source) such that the damage can be repaired prior to the time it would impact the economic well being of the community.

Function II: Normal and ordinary pipeline use, common pipelines in most water systems. All pipes not classified as Function I, III, or IV. The target average break rate of Function II pipelines in 475 year earthquakes should be on the order of 0.03 to 0.06 breaks per 1,000 feet, or less. By "average", we mean that some Function Class II pipelines could have a higher break rate, as long as the overall break rate in the water system is within the target range.

Function III: Critical pipelines and appurtenances serving large numbers of customers and present a substantial hazard to human life and property in the event of failure.

- Pipelines providing water to a minimum of 1,000 service connections including residential, industrial, and business, or other customers; for which there is no redundant supply.
- Pipelines that serve as "backbone" transmission between pump stations and tanks.
- Serious pipeline damage would necessitate very long boil water notice time.
- Pipelines might provide service for any of the facilities indicated below, if the water utility cannot otherwise restore piped water to that facility using its response capability within 24 hours after a rare earthquake:
  - power generating stations and other essential public utility buildings that require piped water supply for operation.
- Function Class III includes sub-transmission and transmission pipes, the failure of which would release high pressure water and/or flood areas that may cause secondary disasters, impede potential emergency recovery, or evacuation of facilities.
- Pipelines servicing facilities otherwise classified as Function II:
  - that are very difficult to restore if damaged.

The target break rate of 12-inch diameter and larger Function III pipelines in 475 year earthquakes should be on the order of 0.004 to 0.008 breaks per 1,000 feet, or less.

Function IV: Essential pipelines required for post-earthquake response and recovery and intended to remain functional and operational during and following a design earthquake.

- Pipelines and appurtenances providing water service to essential facilities that are intended to remain operational during and following an earthquake such as:
  - hospitals and emergency healthcare,

  - emergency shelters,
  - emergency preparedness and response facilities,
  - government essential communication centers,
  - aviation control towers, air traffic control centers, and emergency aircraft hangers,
  - structures critical for national defense.
- Pipelines that provide critical water service to facilities containing extremely hazardous toxic or explosive materials, the release of which poses a serious disaster on the population and surrounding environment.
- Function Class IV includes sub-transmission and transmission pipes, the failure of which would release high pressure water and/or flood areas that may cause secondary disasters, impede potential emergency recovery, or evacuation of those facilities listed that Function IV pipelines provide service.
- Pipelines servicing facilities otherwise classified as Function III:
  - where pipeline damage would disrupt emergency response and operations to those facilities.
  - that are very difficult to restore if damaged.
- Pipelines required to maintain water pressure for dedicated reliable fire suppression systems.
- Pipelines serving as major social and economic centers, the damage of which would significantly impact the state, national, or international social and economic activities.

The target break rate of Function Class IV pipelines in 475 year earthquakes should be less than 0.004 breaks per 1,000 feet.

Exception: pipes of a lower Function branching from one that serves a higher Function should be designed as the higher Function unless it is properly isolated or evaluated as described in Section 3.2.2.5.

In using the above guidance, the authors used judgment when quantifying the numbers of service connections. In some cases guidance was provided from existing building codes. These numbers may be adjusted as determined appropriate for specific cases. In a community with about 1,000 miles of water pipeline, it would be the intention that about 5% (by mileage) of less of all pipes would be Function Class I; about 75% to 85% Function Class II; about 10%-20% Function Class III; and about 1% to 5% Function Class IV.

Community resiliency is dependent upon the ability of social and economic centers to return to normal operating conditions soon after an earthquake disaster. The longer it takes for social and economic centers to recover, the greater the opportunity for the disaster effects to ripple throughout greater parts of the local community and even through the state, country, and for very important economic centers even the world. Inclusion of community resiliency and socio-economic recovery for water pipelines is a necessary extension from normal building codes because pipelines serve a greater portion of the community than a typical building and may be subjected to a broader range of seismic hazards away from the socio-economic facilities they are intended to serve. This concept is not without precedent, as the JWWA (1997) has also included this concept in their guidelines for seismic design of water system facilities.

Pipelines that are difficult to repair and restore include pipes that are deeply buried, located under railroads, rivers, highways, arterial streets, or other facilities that make it difficult to access in normal or emergency conditions. Consideration must be given to pipes under major transportation corridors, as damage to such pipes that leads to shutdown of the transportation corridor might then lead to serious economic impacts and hinder evacuation and rescue operations under emergency conditions.

The four pipe Function classifications were developed to help establish seismic design criteria in relation to a pipes functional use in the water system and to the community. The Function classification concept and definitions of pipes within each Function was initially developed using an analogy with current building code definitions for Occupancy Category and Seismic Use Group with additional definitions included to meet the needs for pipes serving different purposes within a larger water supply and distribution network. The general concept is to establish higher level seismic design criteria for facilities that are more important to the community. Building codes in use across the United States have similar facility definitions as a function of their importance to the community, generally broken into four categories, and are therefore a good measure of the society's expectation on how different types of facilities are to perform during and after an earthquake.

Except for some provisions in the 2003 IBC, current building codes do not govern the design of buried pipelines; although most codes have an implied intent for critical facilities to maintain water service. Building codes govern the design of facilities for which the pipes provide water service and as a result establish the level of seismic risk the society is willing to accept. Building codes consider multiple design levels depending upon the facility use and include provisions for essential facilities, such as hospitals and emergency operations centers, to remain operable during and following a design level earthquake.

Pipelines are essential for providing domestic water supplies to the community. As a result, pipelines are critical for helping communities recover from an earthquake and to help prevent secondary disasters, such as fire and disease, following an earthquake. It is also important to develop consistent seismic design criteria for the community as a whole; that is, on a conceptual level a pipeline need not be designed with greater seismic criteria than the facility(s) in which is serves or for its intended post-earthquake use, or should it be design to a lesser standard than what the community expects. The seismic importance descriptions for very low, normal or ordinary, critical, and essential pipelines are consistent and analogous with building code definitions for building facilities. For example, pipelines servicing facilities defined as essential in the building code are similarly defined as essential in these Guidelines. The authors caution the user not to go "overboard" and call the bulk of the pipelines "essential" or even "critical", in that quite adequate network-wide performance can be met by having the greatest percentage of pipes called "ordinary". To the extent that an owner can show that the community will fare well (at least 90% water restoration within 3 to 7 days after a 475-year or longer return period earthquake), then more of the pipelines could be classified as lower Function Classes, including "very low" Function I.

It is recognized that the pipeline Function concept may be difficult to apply for some pipelines that are part of a system to deliver water throughout large portions of a community. This will be especially true where there are mixed facility types within the distribution area. This is mainly because water systems are developed to provide service within large blocks and not just to a single facility, and many times there will be different facility types of varying importance within the distribution zone. For this reason additional seismic design provisions have been developed considering pipeline redundancy, isolation, continuity, etc. In addition, water systems and facility uses are complicated and it is difficult to identify all variations of use with general guidelines and for these reasons these general Function classification may not conceptually apply to all pipelines; for example if a critical or essential facility can provide a complete self supporting water supply following an earthquake without the need for any domestic supply though normal pipe distribution, then it is possible the pipe seismic design criteria can be altered from the general provisions of these Guidelines. However, in such a case it is recommended that the post-earthquake water supply be clearly evaluated and documented as a part of an emergency response plan prior to determining that these provisions would not apply to the normal pipeline distribution to that facility.

C3.2.2 Earthquake Hazard Return Periods

One of the benefits of using a performance-based approach is that it allows for involvement of the owner/operator in deciding on the performance limit that best reflects its objectives and balance between risk/cost. In the Guidelines we are prescribing the earthquake return period for various Function Classes of pipeline. This removes the owner/operator from the decision process, which might not be desirable in many cases. A return period design of 2475 years for Function Class IV pipelines and facilities may impose a significant financial burden that might, or might not be justified. At any time, the owner may revise the Function Class level of any pipeline subject to meeting the overall performance goal in a cost effective manner; namely that water outages be limited to less than about 3 to 7 days to the vast majority of users, given a rare earthquake. Formalized Benefit Cost Analyses can be done to establish the appropriate Function Classes for groups of pipelines for a specific utility; the procedures for such analyses are described in (Eidinger and Avila, 1999).

On occasion, it has been observed that some seismologists' models can result in extraordinarily high (and sometimes hard to believe) levels of seismic excitation. For example, some seismologist's studies in Northern California predict PGVs exceeding 300 cm/sec for near source forward directivity shaking on the San Andreas Fault for a 975 year recurrence interval. It is important for the user *not* to adopt such high levels of ground shaking when applying these Guidelines, unless they are shown to be *median-based* given all the possible mechanisms of the earthquake source; otherwise, a large and unintended conservatism will be introduced into the design process, resulting in non-cost-effective design. For Function Class IV pipelines, the PGV used for design should rarely much exceed 45 inches per second.

One of the benefits of using a performance-based approach is that it allows for involvement of the owner/operator in deciding on the reliability targets that best reflect its objectives and balance between risk/cost. Table 3-2 presents the design bases for each pipe function.

If the user wishes to conduct a reliability analysis for Function Class IV pipelines, then target reliability might be on the order of 90-95%, given the occurrence of a 475-year return earthquake. Higher reliability targets may be too restrictive, even for Function Class IV pipelines, especially if the user introduces unintended conservatisms into the entire design process.

The authors of these Guidelines did not reach anything like unanimous agreement about the selection of the earthquake return periods in Table 3-2. A vote was taken, and it was nearly equally split, choosing between the following:

- Use a 475-year return period earthquake as being the basis of design for all Function Class II, III and IV pipelines. Then, apply an importance factor (I) of I=1.0 for Function Class II, I=1.25 for Function Class III, and I=1.50 or Function Class IV.

    - Pros. Follows strategy used in typical building codes such as the 1997 UBC. The 2,475-year earthquake is about 1.5 times larger than the 475-year earthquake in high-seismicity locations like much of Los Angeles.

    - Cons. The actual reliability for Function Class III and IV pipelines will differ in high-seismicity Los Angeles as compared to lower-seismicity San Diego, Memphis and other locations.

- Avoid the use of importance factors throughout the Guidelines. Instead, set the design basis for Function Class II, III and IV pipelines as 475-years, 975 years and 2,475 years, respectively.

    - Pros. Avoids the use of I values that ignore areal-specific seismicity issues.

    - Cons. Mixes reliability between the various pipe function classes.

Given the close vote, the authors of these Guidelines would consider it reasonable to design Function Class IV pipelines for 150% of the seismic loading of a 475-year earthquake; and Class III pipelines for 125% of the seismic loading of a 475-year earthquake for any location with reasonably high seismicity (like most of coastal California).

The average return period T is related to P through: $T = -t/\ln(1-P)$, where t is the interval of interest (50 years in Table 3-2). T identifies the average time between seismic hazard occurrences. For practical design purposes, P is sometimes more important than T because engineers are often concerned with a probability of a design parameter being exceeded during an earthquake than considering the time it takes for the hazard to recur. The Return Period T is only presented in Table 3-2 for descriptive purposes because hazard parameters are often presented in terms of T and this parameter is useful for quantifying hazards in terms of a single number. For earthquake hazards, T is more directly related to geological and seismological factors than engineering factors and should be considered in relation to a geologic time scale rather than a facilities useful life.

Defining $t = 50$ years is necessary to present a uniform design basis and is consistent with common engineering practice for design of most facilities. For simplicity and uniformity in design procedures, this value is not recommended to be changed, even if a facility has a longer design life definition. If a different design life is to be evaluated, it is best to re-evaluate the design parameters P in terms of t and T.; for example if a Function II pipeline has a 100-year design life, the earthquake hazard could be presented as having $P = 1 - \exp(-100/475) = 0.19$, or 19% probability of exceedance in 100 years. In this way all pipes in a system could be designed to a uniform hazard-exceedance level regardless of their recognized useful duration.

Actual design lifetimes for pipelines are not well established. While it might be common, for actuarial purposes, to set a design life as 50 or 75 years for a buried pipeline, it should be noted that there are thousands of miles of 100 to 150 year old cast iron pipe still in service in London England. In California, several water transmission pipelines (like the 1925 Hetch Hetchy Aqueduct, the 1927 Mokelumne Aqueduct, etc.) are now approaching ages of 80 to 100 years, and most of them continue to remain in service today; sometimes with updated corrosion control.

C3.2.3 Other Function Class Considerations

It is recognized that Function Classes may be difficult to define for some pipelines that are part of a system to deliver water throughout large portions of a community, especially where there are mixed facility types within the distribution area. This is mainly because water systems are developed to provide service within large blocks and not just to a single facility, and many times there will be different facility types of varying importance within the distribution zone. For this reason additional seismic design provisions have been developed considering pipeline redundancy, isolation, continuity, etc. In addition, water systems and facility uses are complicated and it is difficult to identify all variations of use with general guidelines and for these reasons these general Functions may not conceptually apply to all pipelines; for example if a critical or essential facility can provide a complete self supporting water supply following an earthquake without the need for any domestic supply though normal pipe distribution, then it is possible the pipe seismic design criteria can be altered from the general provisions presented herein. However, in such a case it is recommended that the post-earthquake water supply be clearly evaluated and documented as a part of an emergency response plan prior to determining that these provisions would not apply to the normal pipeline distribution to that facility.

C3.2.3.4 Redundancy

The reliability *R* in a redundant pipe system is determined from:

$$R = 1 - (1 - R_1)(1 - R_2)(\ldots)(1 - R_{L_R}) \qquad \text{[eq C3.1]}$$

where R_{L_R} is the reliability of the L_Rth parallel pipeline. For example, say that a calculation is done that shows that the reliability of one Function Class II pipeline is 85%, given the occurrence of a particular size earthquake. Then, use of Equation C3.1 shows a single redundancy provides a tremendous increase in reliability, for example three similarly-reliable redundant Function II pipes (R=1-(0.15)(0.15)(0.15)=99.7%) would provide an overall 99.7% reliability, a greater level of reliability than normally recommended for Function IV pipes. It is therefore acceptable to reduce the seismic design criteria for truly redundant pipes, provided the minimum seismic design criteria meets or exceeds that of Function II (i.e., those pipes which would not be classified as Function I in Table 3.1 without any redundancy should not be classified lower than Function II). The recommended Function reclassifications in Table 3-3 were established using prudent design limitations by only allowing a pipe Function to be reclassified down one Function level per unit of redundancy. Reliability calculations of this sort can be done using the pipe fragility information provided in ALA (2001).

An alternative to using Table 3-3 is to require one redundant pipe to be designed for its original non-redundant Function classification (say Function Class IV) and all redundant pipes may be designed to provide service as Function I. This might be the case where the two existing pipelines have no seismic design basis, but the new pipeline will. One choice would be to design the new pipe as Function Class II, and retrofit the older two pipes to be Function Class II. Another choice would be to design the new pipe as Function Class IV and leave the original two pipes unchanged. The decision as to which choice to take will depend upon project specific costs.

No matter how much redundancy there is in a retailer's distribution system, a Function Class II pipe is not to be classified as a Function Class I pipeline. Since it is expected that 75% to 90% of all pipes in a water system will be Function Class II pipelines, dropping any material number of them to Function Class I will void the basic performance goal for the water system as a whole, namely to reduce the total level of pipe damage in rare earthquakes to a limited and rapidly (3 to 7 day) manageable level.

Reliability Targets for Water Pipelines

Target reliability levels for various seismic demands give an impression of a state of sophistication in the understanding of expected pipeline behavior and seismic demand definition that at the current time (2005) is imperfect. However, it is clear in the water industry that redundancy is good, some damage is acceptable, and current practices of no-seismic design sometimes leads to unacceptably long water outages post-earthquake.

In order to establish a rational design approach, one must set some performance target, and then use available analytical and empirical methods to try to achieve that target. Unlike the oil and gas industry, where pipe failure sometimes leads to large environmental and economic consequences, it has been common practice for many decades in the water industry to assume that leaking water pipes can be readily fixed without undue consequences. With the exception of long outage times and loss of water for fire service after rare earthquakes, this philosophy has mostly served our communities well.

C3.2.3.5 Branch Lines and Isolation

If a Function IV pipeline has a branch pipeline, then that branch pipeline also needs to be designed as a Function IV pipeline. For example, an Essential (IV) pipeline with a lateral serving a fire hydrant may be made non-functional if the hydrant lateral breaks. Since by definition the IV pipeline is to remain in service without interruption, the hydrant lateral also needs to be designed as a IV. Alternately, a valve can be placed at the interface of the essential pipeline and its branch pipeline, and the branch pipeline designed to a lower function, as long as the owner accepts that it may take some time to close the valve and isolate the damaged lateral, and that this amount of time is acceptable within the overall context of post-earthquake response and recovery.

In order to set the target post-earthquake performance in a cost effective manner and in consideration of how typical water systems are operated, we make the following observations.

- Water systems are usually divided into multiple and separate pressure zones. Pressure zones are usually hydraulically separated from other pressure zones, such that a pipe break in one zone does not directly affect the pressures and flows in another zone.

- The post-earthquake performance of a pressure zone is highly correlated to the "break rate" of pipelines within a pressure zone. The post-earthquake performance of a pressure zones will also depend on concurrent damage to tanks, pump stations, loss of electric power, which are all readily mitigated and are outside the scope of these Guidelines.

- A "break" is defined as the complete separation of a pipeline, such that no flow will pass between the two adjacent sections of broken pipe.

- A "leak" is defined as a small leak in a pipeline, such that water will continue to flow through the pipeline, albeit at some loss of pressure and flow rate being delivered, with some flow being lost through the leak. Leaks can include pin holes on the pipe barrels; very minor joint separations on segmented pipes; very small splits in large diameter steel transmission pipes, etc.

C3.2.3.7 Damage and Post Earthquake Repair

In establishing acceptable post-earthquake system performance, one needs to establish the flow rates to be delivered and the recovery time, which is correlated to pipe break rates.

It is generally found to be cost effective to plan (in urban areas) for only winter time (wet season) flow rates for response and recovery after rare earthquakes. In California, this would be the maximum of the daily flows for the months of December, January, February and March. This implies that a few percent of economic activity (outdoor irrigation uses) may have to be curtailed for the few days post-earthquake until complete system repairs can be made. In agricultural areas reliant largely on irrigation, this criteria would be modified depending on the drought sensitivity of crops, etc.

To establish what constitutes "acceptable" performance of a pressure zone after an earthquake, we make the following generalized assumptions about network connectivity, break and leak rates, and normalization. We normalize pipe breaks and leaks into "equivalent 6-inch diameter breaks".

The intent of these Guidelines is to assure a reasonably low rate of water pipeline damage throughout a water utility system, such that about 90% of customers in a system can be restored with piped water service within about 3 to 7 days after a rare (475 year return period) earthquake. This is a primary service restoration target that can be adopted by a water utility.

To achieve this level of performance, an acceptable damage rate will be about 0.03 to 0.06 breaks per 1,000 feet of equivalent 6-inch diameter pipe. The following analysis explains how this criteria can be quantified for various types of networked pipe systems. By performing this type of analysis and confirming that the service restoration target is met, the owner may lower the Function Class of particular pipelines to as low as Function Class I.

The number of equivalent 6-inch diameter breaks is calculated as follows (example, in a pressure zone with 4-inch to 60-inch diameter pipe):

$$E_{be} = \sum B_{eq} + \left(L_{eq} * 0.018\right) \qquad \text{[Eq C3-2]}$$

where

$$B_{eq} = \sum_{d=4}^{d=60} b_d * \frac{d^2}{36} \qquad \text{[Eq C3-3]}$$

$$L_{eq} = \sum_{d=4}^{d=60} l_d * \frac{d^2}{36} \qquad \text{[Eq C3-4]}$$

b_d, l_d = Number of breaks and leaks of diameter d
d = nominal pipe diameter, inches

and the coefficient 0.018 represents a 1-inch diameter leak (such as typical for a service line connection failure, or a leak due to corrosion).

The number of breaks and leaks (b_d and l_d) can be calculated using fragility formulations such as in ALA (2001), coupled with a suitable description of the seismic hazard and geotechnical ground failures, such as in Eidinger and Avila (1999); or by any other suitable method. Pipes designed in accordance with these Guidelines for Function Classes II, III or IV will have materially improved fragilities and much lower repair rates than corresponding Function Class I pipelines.

Once E_{be} is established, the hydraulic performance of a pressure zone can be estimated using the following steps.

First, estimate the normalized equivalent break rate X_{br} per 1,000 feet for the pressure zone as follows.

$X_{br} = \frac{E_{be} * 1000}{L}$, L=length of pipe in zone, in feet

Depending on the size of a pressure zone, a single 6-inch diameter pipe break could have from very minor to substantial impact on overall system performance. In a large pressure zone (one with more than 100 miles of pipe), the effect of a single 6-inch break would be similar to the effect of opening one or two fire hydrants – there will be a localized pressure drop, but most customers will not sense any appreciable change in flow and pressure. However, a single 6-inch break in a small pressure zone (one with less than 10 miles of pipe), the impact of a single 6-inch break will be more significant.

In the post-earthquake environment, the percentage of customers with water will vary significantly immediately post-earthquake, when leaking and broken pipes are actively flowing; and a few hours and up to a day later, once the water utility acts to isolate the bulk of the pipe damage. Figures C3-1 and C3-2 illustrate these two conditions.

To set the target performance goals, we make the following assumptions.

- A typical water utility will want to be able to deliver water to at least 90% of all customers within 3 to 7 days following an earthquake.

- A typical water utility will be able to isolate most of the leaking and broken pipes within 1 day or so.

Using Figure C3-2, an "acceptable" normalized break rate X_{br} is about 0.03 to 0.06. For X_{br} of 0.03, about 90% of all fire hydrants will be serviceable immediately after the earthquake. For X_{br} of 0.06, about 65% of fire hydrants will be serviceable immediately after the earthquake. For X_{br} of 0.06, about 83% to 91% of all customers will have water once the leaking and broken pipes are valved out.

For X_{br} of 0.20, performance immediately post earthquake will be very poor (just 15% of hydrants with water).

By integrating over all pressure zones, and considering its own emergency response capability, a water utility can establish system wide restoration times. A detailed analysis could also be performed by a utility for any specific situation on hand to refine the data in Figures C3-1 and C3-2 and to establish Function Classes for all its pipelines.

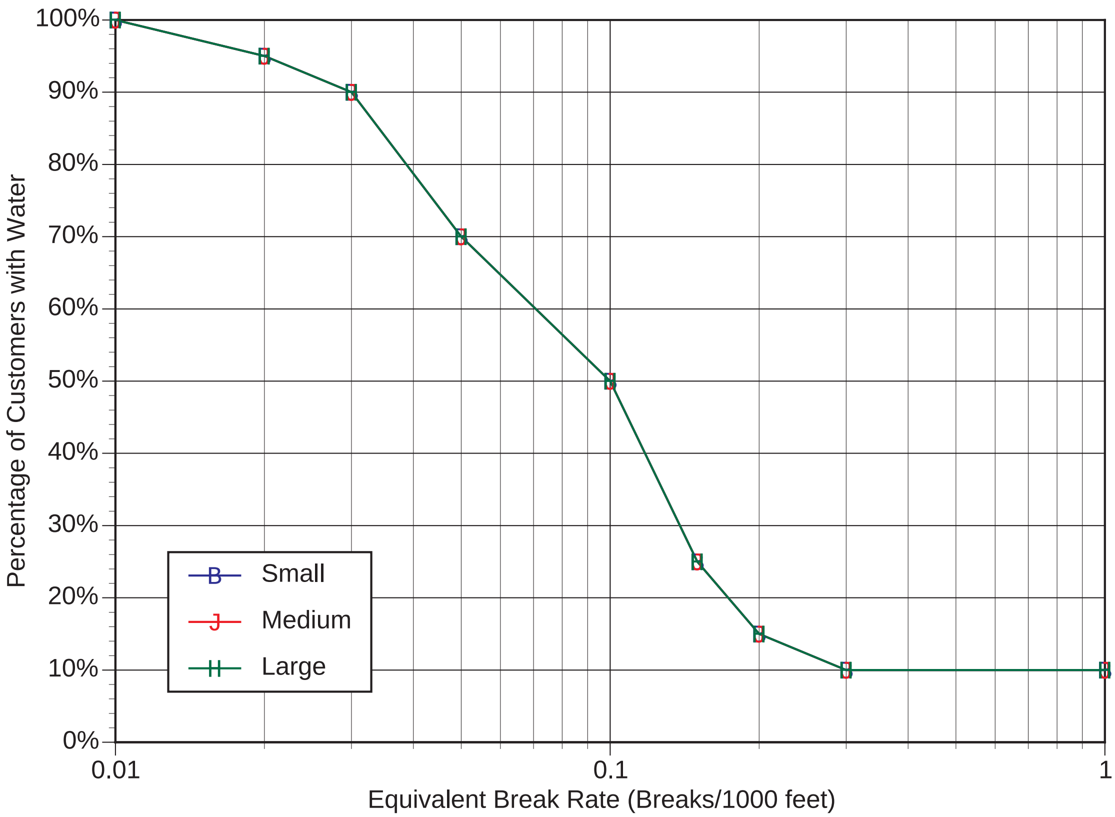


Figure C3-1. Customer Service, Before Leaks and Breaks are Isolated

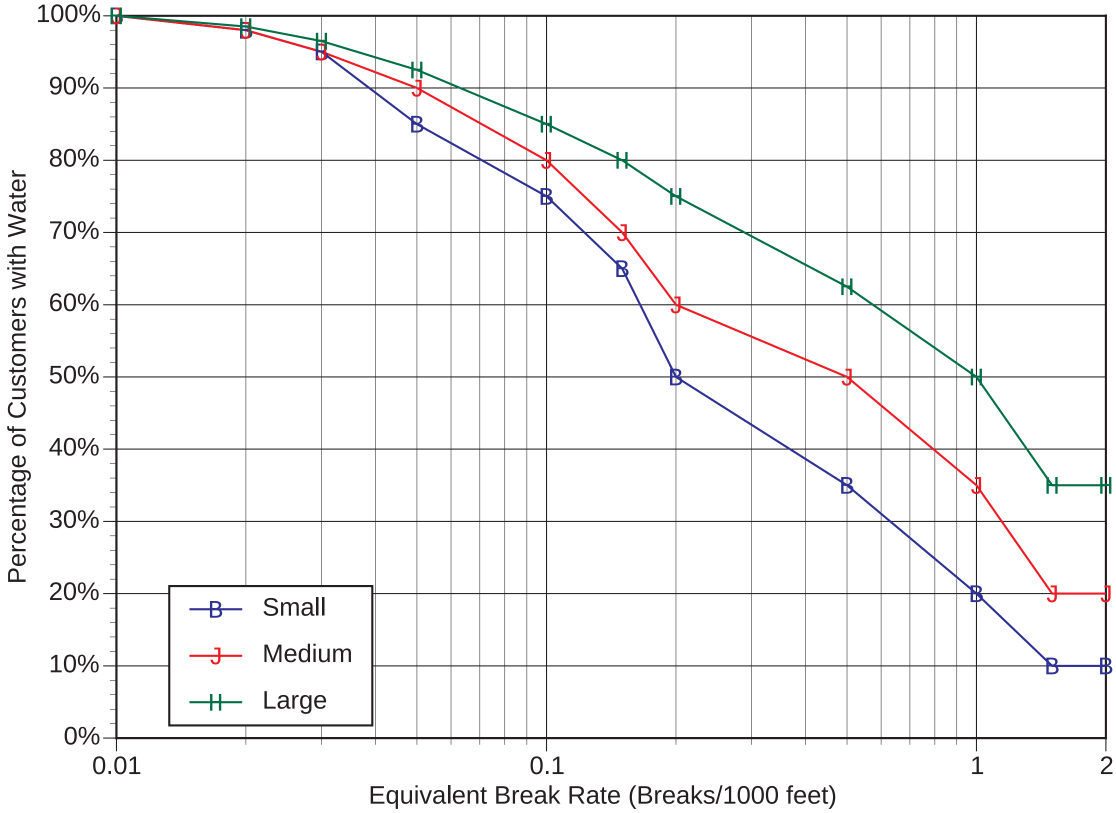


Figure C3-2. Customer Service, After Leaks and Breaks are Isolated

C3.3 Other Guidelines, Standards and Codes

We examined various building codes from around the world to see what guidance in these codes might pertain to the seismic design of buried pipelines. The following highlights our findings.

C3.3.1 2003 International Building Code

IBC 1604.5: Importance Factors

- Importance factor Category IV, I_E = 1.5, for water treatment facilities required to maintain water pressure for fire suppression; public utility facilities required as emergency backup facilities for Category IV structures including hospitals emergency healthcare, fire rescue and emergency support, emergency shelters, emergency preparedness and response facilities, aviation control towers, air traffic control centers, and emergency aircraft hangers, structures critical for national defense. These are Seismic Use Group III.

- Importance factor category III, I_E = 1.25, for all other water treatment for potable water and other public utility facilities not required for fire suppression. These are Seismic Use Group II.

IBC 1616.2: Seismic Use Group

- Seismic Use Group II (Importance Category III) are those for which the failure would result in substantial public hazard.
- Seismic Use Group III (Importance Category IV) are those essential facilities that are required for post-earthquake recovery and those containing substantial quantities of hazardous substances.

IBC 1622.1.3. Add Section 9.14.7.9 to ASCE 7 for Buried Structures:

- Defines pipes as buried structures that are either Seismic Use Group II or III (as indicated above) as requiring to be identified in a geotechnical report.
- Requires flexible couplings be provided where changes in support system, configurations, or soil conditions occur.

General assessment of IBC provisions

The authors of these Guidelines do not feel that the provisions of the IBC should be applied to buried water pipelines. This said, we observe the following:

- The IBC attempts to require seismic design of pipes to ensure consistency in seismic design to provide an adequate community response following an earthquake.
- The IBC establishes that all water system and utility components, including pipes, are considered as critical in that any failure poses a substantial public hazard.
- Terminology is not correct in that pipes are identified as “water treatment.”
- The IBC seems to attempt to place pipeline design under jurisdiction of building officials approvals. This may cause great difficulty in that pipe systems are not designed and constructed in a similar manner as other structures. Pipes are also generally in the public right-of-way where the code generally will limit public officials from having jurisdiction. Thus, there is an inherent conflict here.
- The IBC does not consider a water system as a whole in that the system may have adequate redundancy for some pipes to not be considered critical.
- IBC seismic ground motion requirements are not consistent with that needed for buried pipe design.
- The IBC does not address ground deformation hazards of any type as related to pipelines.

- The use of term "back up facilities" in identifying use category is misleading in that water supply networks are not backup facilities but essential supply (and/or support) facilities for other essential facilities and they must be able to continually function together.

- The IBC clearly intends to have pipelines design to withstand earthquake effects. The IBC follows provisions of ASCE 7-02 and therefore intends to have essential facilities maintain functionality during and following an earthquake. Therefore, the code intends to maintain pipeline system functionality.

C3.3.2 ASCE 7-02 and 7-05

There is no intent in either ASCE 7-02 or 7-05 that these documents should be applied to the general design of buried pipelines.

Section 1.5: Nature of Occupancy

- Clearly identifies that occupancy is related to structures other than building structures, which includes pipe structures.

- Commentary clearly identifies that the purpose is to "*Improve the capability of essential facilities and structures containing substantial quantities of hazardous materials to function during and after design earthquakes*." This is achieved by including an importance factor I to reduce structure ductility demands in combination with stringent drift limitations.

- Category IV structures include water storage facilities and pump structures required to maintain water pressure for fire suppression and other public utility facilities required in an emergency. These are defined in the commentary as buildings and other structures intended to remain operational in the event of extreme loading and include ancillary structures required for operation of Category IV facilities during an emergency. These are Seismic Use Group III.

- Category III structures include other public utility facilities not included in Category IV. These are defined in the commentary as buildings and other structures representing a substantial hazard to human life in the event of failure. These are Seismic Use Group II.

9.1.3: Seismic Use Group

- Seismic Use Group II defined for Occupancy Category III.

- Seismic Use Group III defined for Occupancy Category IV.

9.1.5: Occupancy Importance Factors

- Seismic Use Group II, I = 1.25.

- Seismic Use Group III, I = 1.5.

General assessment of ASCE 7-02 provisions

- Attempting to require seismic design of pipes to ensure consistency in seismic design to provide an adequate community response for essential facilities following an earthquake. This is understood through use discussion in the commentary; however, this is not as clearly defined as in the IBC.

- Pipes defined as essential facilities could be improved with better terminology. Table 1-1 only identifies water storage facilities and pump structures needed to supply water pressure. This in its literal interpretations is limited to tanks, reservoirs and pump stations. Nothing in ASCE 7-02 (or 7-05) is intended to cover the design of buried water pipelines; except that utility connections should have flexibility if needed where they attach to buildings.

- Does not consider the water system as a whole.

- IBC seismic load requirements are not consistent with that needed for buried pipe design.

- There are no existing codes, standards, or guidelines addressing the seismic design of pipeline networks either from a systems point of view or a strength point of view (with the exception of these Guidelines).

- The ASCE 7-02 is not as specific as IBC 2003 in identifying pipelines falling under code provisions. According to ASCE 7 code members, they did not intend ASCE 7 to cover buried water pipelines.

C3.3.3 1997 NEHRP provisions.

- Same description as provided for IBC 2000. IBC 2000 is essentially the same as the 1997 NEHRP provisions

C3.3.4 1997 Uniform Building Code (UBC)

UBC 1626.1 Purpose:

- The CBC and LABC identify limitations on the seismic provisions to indicate it is intended to only safeguard against major structural failures and loss of life, but not to limit damage or maintain functionality.

UBC 1629.2: Occupancy Categories & Importance Factors

- Category 1, Essential facilities, include function of tanks or other structures containing, housing, or supporting water or other fire suppression material or equipment required for the protection of Category 1, 2, or 3 structures (Cat. 2 = hazardous facilities).

- Importance factors I=1.25, I_p=1.5.

- Category 3, Special occupancy structures, include function of structures and equipment for public utility facilities not included in Category 1 or 2 and required for continued operation.

- Importance factors I=1.0, I_p=1.5 (for life safety systems).

- Importance factor I is used for structural systems and I_p for elements of structures, non-structural components, and equipment supported by structures.

General assessment of UBC provisions

- There is a direct conflict in the UBC code provisions with the purpose clearly stating there is no intent for the UBC provisions to maintain functionality while the occupancy category identifies utilities required for continued operation. If the code is developed to allow loss of functionality then it will not have continued operation.

- The UBC clearly implies intent to cover the design and construction of pipelines for the purpose of protecting certain types of facilities in connection with having an adequate community response to an earthquake disaster. However, even the type of protection, such as fire, is not clear, and the code conflict described above further confuses any level of interpretation of how a water system is intended to perform.

- There is nothing in the UBC that provides adequate seismic design criteria for a buried pipeline.

C3.3.5 1997 JWWA Guidelines

- The JWWA guideline has sections specifically describing the seismic design criteria for buried water pipelines. This is probably the first industry-group-based document in the world developed for the purpose of identifying guidelines for the design and construction of buried water pipelines. The predecessor of these guidelines is a document prepared by the Kubota ductile iron pipeline company.

- Design water pipes using two-level seismic ground motion system, Level 1, L1, and Level 2, L2. See review on ground motion parameters (below) for more information on L1 and L2. The Japanese do not define L1 and L2 motions with specific return periods; L1 would be comparable to a 100 to 200-year return period motion; L2 would be comparable to a deterministic M6.8+ earthquake that strikes directly beneath a city, such as the 1995 Kobe Great Hanshin earthquake.

- Facilities are given a seismic Rank A or B identifying its relative importance. A is more important than B. Water Systems must rank their own facilities (pipelines in this case) based on the water facility location with respect to other social and economical facilities.

- Rank A: facilities with a high level of importance
- Rank B: other facilities

- Definition of Rank A facilities:
  - Facilities (water system facilities or other owned facilities) that possess the ability to generate serious secondary disasters.
  - Water facilities located upstream of water supply system (note the JWWA places higher importance of upstream facilities than distribution facilities – this is presumably a result of devastating effects in Kobe resulting from supply source damage. (In these Guidelines, we place such importance only if damage would cause loss of raw or treated water supply to a large community and that community does not have at least 30 days of local terminal storage; otherwise, the raw water pipeline can treated as a lower classification)
  - Main water facilities which do not have backup facilities
  - Feeder mains to important facilities (water or other facilities). JWWA commentary defines important facilities as evacuation facilities, hospitals, transformer stations, waste incineration plants, and wholesale markets which may greatly affect the community's social or economical activities. (These Guidelines similarly provide more stringent design for non-redundant pipes that directly serve critical care facilities.)
  - Main water facilities which are difficult to restore if damaged. JWWA commentary defines difficult to repair as pipelines under railroads or rivers, pipelines which are deeply buried, and main facilities which are built near active faults. (The authors of these Guidelines concur that non-redundant pipelines should have superior design where they cross under highways or other difficult-to-repair locations.)
  - Facilities (water system or other government or social facilities) used for information gathering during a disaster.

- Seismic Design Criteria
  - L1 ground motion effects on Rank A facilities: <u>no damage</u>

- L2 ground motion effects on Rank A facilities: light damage but remains functional with no severe impact on human life. (The authors of these Guidelines doubt that this can be achieved in every case for every water pipe, and achieving a 95%+ reliability give the occurrence of a L2 earthquake will usually be satisfactory).

- L1 ground motion effects on Rank B facilities: light damage and may not be functional, but quickly restored to service

- L2 ground motion effects on Rank B facilities: damage may be sustained but the water system able to remain functional. (The authors of these Guidelines specifically allow that some damage to regular (especially Function Class II) pipelines is acceptable, as long as the damage can be managed in an acceptably short time).

C3.3.6 ASCE 1984

"Guidelines for the Design of Oil and Gas Pipelines," ASCE Committee on Gas and Liquid Fuel Lifelines, 1984.

- Purpose: to present current (1983) state-of-the-practice of earthquake engineering for oil and gas pipeline systems as a unified set of guidelines.

- Oil and Gas pipelines are considered essential facilities due to their need for energy at critical facilities, transportation for emergency response, etc. and because they contain hazardous chemicals and materials detrimental to human life and the environment.

C3.3.7 ASCE-ASME 2001

"Guide for the Design of Buried Steel Pipe," Joint ASCE-ASME Task Group on Buried Pipe Design, June 2001.

- Purpose: to develop design provisions for the evaluation of the integrity of buried pipe for a range of applied loads.

- Covers welded steel pipe.

C3.3.8 PRCI 2004

"Guidelines for the Seismic Design and Assessment of Natural Gas and Liquid Hydrocarbon Pipelines," Pipeline Design, Construction and Operations Technical Committee of the Pipeline Research Council International, Inc., October 2004.

- Purpose: to present current (2004) seismic guidelines for the design and assessment of natural gas transmission systems.

- Intended to be an update of the ASCE 1984 guidelines for buried pipelines.
- Covers welded steel pipe.

C4.0 Earthquake Hazards

Earthquake hazards are often described in terms of PGA, PGV and PGD. Depending on the item to be assessed, spectral acceleration, magnitude (as a proxy for duration) and other indices may be used to describing the earthquake hazard.

It is up to the user to decide when a geosciences expert should be retained to establish the earthquake hazards. All the models presented in the Guidelines and Commentary are based on relatively simple-to-use procedures that may not always be suitable for the project at hand. A geosciences expert should often be retained for all Function Class IV installations, sometimes for Function Class III, and occasionally for Function Class II. The geoscience models in the Guidelines and Commentary should be suitable for conceptual design of any pipeline, and might be refined for final design.

Table 4-1 present the common earthquake hazards considered in design. There are several other earthquake hazards presented in Table C4-1 that are know to cause pipeline damage in many past earthquakes.

Hazard	Earthquake Parameters	Obtain from:	Geotechnical Parameters
Transient Ground Movement			
Impedance boundaries	pga, pgv	PSHA	Soil/rock interface conditions, depth, V_s
Topographic amplification	pga	PSHA	Topography
Basin edge	pga	PSHA	Basin subsurface geometry, soil & rock properties, source distance
Ground Oscillation	Acceleration time history	PSHA, site specific analysis	Soil profile, strength, V_s, groundwater
Permanent Ground Movement			
Shear deformations	pga or pgv	PSHA	Soil type, strength, thickness, groundwater
Ridge shattering	pga	PSHA	Topography, rock/soil properties, rock fractures & orientation

Table C4-1. Earthquake hazards and parameters needed for pipeline design

C4.1 Transient Ground Movement

The PGA, PGV and spectral shape quantified values that are obtained from the USGS web site (as of 2005) are based on attenuation models that consider magnitude and source distance. These will usually be adequate for most situations.

In special circumstances, local topology and soil conditions can create situations where transient ground movements may be amplified. The following paragraph describes some of these situations.

Local topography such as valleys and ridges may modify the ground motions. Ridges can amplify shaking by factors of 1.5 to 2 (Bouchon, 1973). Valleys and canyons can create reflected and refracted amplifications shadow zones. Waves propagating from relatively hard rock into large sedimentary basins may generate surface wave amplifications along the basin edges (Somerville and Graves, 1993). Waves may also become trapped within the basin creating an oscillation effect as they resonate within the basin. Large strains can be generated along edges of sediment filled valleys or basins where there are significant impedance boundaries. On a smaller scale, where firm soils overly weaker soils susceptible to strain induced strength loss or liquefaction, transient ground oscillation may manifest with large localized horizontal transient movements developing large strains amplitudes near contacts with more competent ground.

C4.2 Liquefaction

Cohesionless soils are more predominately susceptible to liquefaction, but some cohesive soils having less than 15% of grain size less than 0.005 mm, a liquid limit less than 35, and water content greater than 90% of the liquid limit are considered susceptible to liquefaction. The liquefaction susceptibility generally decreases with increasing fine grained cohesive particles. Loose silty soils with little cohesion are potentially liquefiable. Gravelly soils are also potentially liquefiable.

By definition, liquefaction occurs when the pore water pressure equals the overburden pressure. During shaking, pore pressures may increase gradually or rapidly from their initial static pressures depending on the seismic induced shear strains. Pore pressures can increase from their static values without developing full liquefaction.

The loss of soil shear strength can lead to large ground strains resulting from permanent or transient ground movements. Shear strength reduction may lead to down slope permanent movements. Transient movements are manifested through ground oscillations. Permanent ground movements from a few millimeters to several tens of meters are manifested through lateral spreading, flow failure and settlement. In addition to the lateral spread, flow failure, and settlement described in section 4.2, soil strength loss from pore pressure increases or complete liquefaction can lead to bearing failures. Bearing is a greater problem for above ground pipes supported on foundations and for pipes extending under structures and roadways.

A major goal of these Guidelines is to help water utilities achieve a seismically-sound water system pipeline network in a cost-effective manner. In general, mitigation for long return period earthquakes will not be economically justifiable, especially in terms of retrofit. A 2,475 year-return period earthquake (same as 2% in 50 year) is sufficiently long such that the potential benefits of retrofitting all buried water pipelines in a network (very expensive) would provide less benefit than the initial cost to the local society (a benefit cost ratio much less than 1).

The key to achieving improvements in seismic performance for long-return-period earthquakes is to identify relatively small portions of the system at greatest risk and with most importance and implement protective measures at these locations. In this way, improvements in safety and performance under low probability, high consequence events can be attained at a relatively modest cost, with an associated acceptable increase in installation expense.

For much of Eastern United States, the 2,475-year-return-period earthquake will result in PGA values of 0.3g or higher using the methodology described in Chapter 4. In contrast, the 475-year-return-period earthquake might result in PGA under 0.10g. Because PGAs less than 0.10 g are not likely to trigger liquefaction or landslide activity, the 475-year-return period earthquake will not result in PGD, thereby bypassing the need to install pipelines with sufficient capacity to resist the effects of liquefaction.

It is well recognized that liquefaction-induced PGD, especially lateral spread, is one of the most pervasive causes of lifeline damage during earthquakes (T. O'Rourke, 1998). To protect against the most serious effects of liquefaction, which would occur for long-return-period earthquakes in the Eastern US and other locations with low frequency in seismic activity, it is necessary to strengthen pipelines most exposed to the risk of liquefaction. To control and limit the cost associated with this strengthening, it is necessary to focus on areas where soil deposits are most susceptible to liquefaction and resulting PGD effects. In this way, it is intended that all zones of the USA where moderate to severe earthquakes are credible (like Memphis, Charleston, St. Louis, Salt Lake City, etc.), are afforded a reasonably cost-effective measure of earthquake reliability of the water pipeline network.

To achieve an improved measure of earthquake reliability under these conditions, two approaches are recommended, associated with 1) estimates of lateral PGD using Eq. 4.10, and 2) the use regional liquefaction maps. Both these approaches are described in Sections C4.6. The approach that is most consistent with the data, technical expertise, and goals of the owner/operator should be used.

C4.3 Permanent Ground Movement

In addition to surface fault rupture, other tectonic deformations may result from general warping of the ground, compression folding, and ground extension. In general these deformations occur over relatively large distances with little strain. However, there are some specialized conditions of potential concern to pipelines. Folding may cause sympathetic slip along bedding planes, pre-existing rock fractures and joints, or other faults. Tight folds can fracture and develop large local ground strains. Extensional features may cause ground fractures and grabens with horizontal and vertical offsets.

Permanent shear deformations result in relatively flat ground when cyclic inertial loads exceed the reduced effective soil strength. These deformations may be associated with saturated or unsaturated relatively weak fine or coarse grained soils. These movements are similar, but usually with less deformation, to lateral spread deformations is liquefied soils.

Ridge shattering typically involves deformation and disturbance of loose surficial soils and rock overlying more competent rock from the amplified ground motions at the top of steep ridges. These types of permanent movements generally are not of concern to pipelines, above or below ground. However, in instances where there are large continuous vertical or near vertical fracture planes in a ridge, the amplified ground motions may cause large out-of-phase movements at the top of ridge. The out-of-phase movements result in large transient and permanent ground strains and may allow the shattering to extend along deeper planes in the rock. The differential out-of-phase movements can allow grabens to form as rock wedges slip downward when the ridge separates along the weak planes. The violent shaking can also cause slides to occur along the steep slopes.

C4.4 Seismic Hazard Analysis

A deterministic seismic hazard analysis (DSHA) considers the effects from a particular earthquake scenario on a pipe. In a deterministic seismic hazard evaluation a particular fault would be selected and assumed to generate an earthquake of certain magnitude from which ground motions along a pipe alignment would be estimated. The earthquake scenario is determined based on a judgment that a particular earthquake(s) may pose the most significant hazards on the pipe when an earthquake is generated from a particular location on that earthquake source. A DSHA is relatively simple to carry out and easier to understand than probabilistic methods, but it cannot adequately account for uncertainties in the evaluation and does not account for the risks associated with the accumulation of all seismic sources potentially affecting the pipe. Using a DSHA approach may require multiple scenarios be evaluated for a single pipe and for different pipe located through different parts of a single water supply and distribution system. Water systems located in highly seismically active regions would usually necessitate multiple deterministic scenarios. The different scenarios generally have different recurrence intervals and lead to inconsistent results in that the pipeline design.

A probabilistic seismic hazard analysis (PSHA) simultaneously considers the effects from multiple earthquake source hazards on a pipe and the probabilities of a likely range of magnitudes over the length of each seismogenic source. A PSHA is more difficult to carry out and understand than deterministic methods, but it does account for uncertainties in the evaluation and can account for the risks associated with the accumulation of all seismic sources potentially affecting the pipe. PSHA does not present simple results relating ground motions to a particular fault at a distance from a pipe alignment. Instead PSHA results are an accumulation of relative contributions of all sources considered in the evaluation. A mean and mode magnitude and distance and all relative source contributions can be presented through deaggregation of the PSHA.

An understanding on how the differences in DSHA and PSHA will affect pipeline design results can be developed through an example. Consider a long pipeline crossing two faults with each having maximum probable earthquake similar to a characteristic earthquake of magnitude M6.5. One fault has the M6.5 recurring on an average of every 300 years and the other on average of 3,000 years. A DSHA approach would evaluate the hazard by estimating where the epicenters may be located and assuming the two M6.5 earthquakes pose the same total risk to the pipe. Thus DSHA ground motions and other earthquake hazards would be similar at similar distance for each earthquake scenario. A PSHA would account for each earthquake recurrence interval, the probability of the epicenter being located anywhere along the faults, and hazards associated with other nearby faults that may potentially affect the pipe. PSHA deaggregation would identify the relative contributions from the different sources, which identifies the dominant magnitude at dominate distances from the pipe for the different sources. The PSHA deaggregated magnitude and distance would be similar to that from the DSHA for the two faults under discussion because they have similar characteristic and maximum probable properties. However, the DSHA results would likely provide lower ground motions and a total reliability level that these motions would be exceeded in any given earthquake could not be adequately estimated. This example shows how a DSHA can underestimate the earthquake hazards. It is also difficult to design the pipe with a uniform approach using a DSHA because the probability of exceedance levels for each scenario earthquake are different. The earthquake having the longer recurrence interval poses much less risk to the pipe, but the DSHA does not account for this fact. The deterministic earthquake parameters needed for some hazard assessments shown in Table 4-1 can not be determined from the PSHA, but can be adequately evaluated through deaggregation. Thus, for pipeline design, a PSHA allows for a uniform probability of exceedance evaluation and is recommended for use in these Guidelines. A DSHA serves a useful in a water system evaluation and is recommended for use in addition to a PSHA to ensure a system can adequately survive known earthquake hazard scenarios.

An example where a DHSA might be better than a PSHA is when a particular pipeline is located near and about mid-way between two active faults. This is often the case for pipelines located near San Jose, California, where a magnitude 7+ event on the Hayward fault, or a magnitude 7.8+ event on the San Andreas fault might both occur within the planning time horizon. In such a case, duration-susceptible phenomena, such as liquefaction, might be best characterized assuming the deterministically worst event (San Andreas M 7.8), even if the Hayward M7+ event is somewhat more likely to occur first. The user is thus cautioned that the de-aggregation plot in Figure 4-3, is useful to pick out the magnitude/distance event that contributes most to the overall hazard level, but still the user may wish to design the pipe for multiple magnitude/distance earthquake scenario events.

C4.4.1 Probabilistic Seismic Hazard Analysis (PSHA)

Technologies for performing a PSHA have advanced tremendously over the past few decades (McGuire, 2004), much of the advancements have been developed through the United States Geological Survey (USGS). Assembling and processing data and developing a new program to perform a PSHA are a very difficult, time consuming, and expensive processes. The USGS has developed an interactive deaggregation web page for performing site specific PSHA, which is accessible on the World Wide Web at: http://eqint.cr.usgs.gov and is recommended for use with these Guidelines.

The PSHA is performed assuming all sites to be rock with an average $Vs = 760$ m/s in the uppermost 30 m, corresponding to ground class B/C as defined in the next section. More detailed information regarding methodologies used and assumptions made by the USGS in performing the PSHA are available on the USGS web page. McGuire (2004) also provides a very good description of PSHA. A detailed description of a PSHA is beyond the scope of these Guidelines.

Figure 4-1 shows that seismograms presented as time histories of acceleration can also be obtained. These will not be described or used as part of these Guidelines. A PSHA can be performed for a 1, 2, 5, 10, 20, and 50% probability of exceedance in 50 years to evaluate PGA and spectral accelerations at frequencies of 0.5, 1.0, 2.0, 3.33, 5.0, and 10 hz. To obtain the necessary parameters shown in Table 4-1 for Function II, III, and IV pipes, only PSHA for 2, 5, and 10% probability of exceedance in 50 years for PGA and 1.0 hz spectral acceleration will be performed.

For purposes of seismic evaluation, a building site is generally considered to have approximate dimensions of a standard city block. Pipelines generally cover much large distances and the concept of a site may not be applicable. Therefore, pipelines should be broken down into several sites for PSHA. The PSHA results do not change significantly over short distances and therefore only a limited number of sites need be evaluated for each pipe. The total number of site evaluations is dependent upon the total pipe length and number of seismic hazards the pipe crosses. It is recommended to perform at least two PSHA for each pipe, one at each end. If the results vary significantly on each end of the pipe, several additional sites need to be evaluated along the pipe alignment to ensure appropriate design values are obtained. Consideration should also be given to performing a more detailed grid of sites near fault crossings, in landslide hazard zones, in liquefaction hazard zones, and in areas suspected of having large shear deformations.

Table 4-1 shows that use of *PGA*, *PGV*, *M*, *R*, spectral response, and an acceleration time history, provides a complete set of parameters for a pipeline seismic hazard evaluation. All of these parameters may not be needed for different pipes, but a uniform methodology for obtaining the parameters needs to be identified. Descriptions to this point have shown how to determine all parameters except for *PGV*. The USGS does not provide a PSHA for PGV_B and therefore this value can not be determined directly from the procedures presented. *PGV* is closely related to the spectral velocity at 1 hz, SV_1 (Naeim and Anderson, 1993, Newmark and Hall, 1982). Applying the relationship between SV_1 and the spectral acceleration at 1 hz, SA_1, PGV_B can be estimated from Equation 4-1.

C4.4.1.1.1 Getting PGA and PGV

The PSHA procedures listed in the Guidelines to establish site-specific PGA and PGV values rely on country-wide seismic hazard analyses at grid point for soil class B rock-like conditions and simplistic conversion tables to consider site-specific soil conditions.

The user is always allowed to use site-specific methods to establish the hazard at particular locations.

Peak ground motion parameters include peak ground acceleration and peak ground velocity and may be determined from site specific evaluations of the maximum considered earthquake using mean ground motions for 10%, 5% or 2% chance of exceedance in 50 years for Function Class II, III and IV pipelines. The peak ground velocity PGV_B can be estimated from:

$$PGV_B = \left(\left(\frac{386.4}{2\pi} \right) SA_1 \right) / 1.65 \quad \text{(PGV in inch/sec, } SA_1 \text{ in g)}$$

Peak ground acceleration, spectral acceleration and other ground motion properties for soil class B sites can also be estimated from the United States Geological Survey web site, http://eqint.cr.usgs.gov/eq/html/deaggint2002.html.

There are at present no widely available maps in building codes that provide PGV levels. The user can always prepare a site specific study to establish these levels. The tables in Section 4.2.4 provide a simplified way to adjust the PGA values to PGV.

Another lookup table to convert PGA to PGV is provided in Table C4-2. In order to use this table, the user must define the distance from the causative earthquake to the pipeline site, and the magnitude associated with the causative earthquake. For many sites, the total seismic hazard will be the sum of earthquakes from varying causative earthquake sources, so the lookup in Table C4-2 may have to be performed for each source.

There are other ways to convert the PGA values to PGV, possibly without having to consider M. These simplified methods have the merit of being "more simplified" but "less accurate".

Soil Classification	Ratio of Peak Ground Velocity (cm/sec) to Peak ground Acceleration (g) Source-to-Site Distance (km)		
Moment Magnitude	0-20 km	20-50 km	50-100 km
Rock: A, B			
6.5	66	76	86
7.5	97	109	97
8.5	127	140	152
Firm Soil: C, D			
6.5	94	102	109
7.5	140	127	155
8.5	180	188	193
Soft Soil: E, F			
6.5	140	132	142
7.5	208	165	201
8.5	269	244	251

Table C4-2. Alternate PGV to PGA Relationships

C4.4.2 Design Level PGA and PGV Values

For high seismicity parts of California, the 2/3 factor in the IBC (2000) very approximately converts the 2% in 50 year motion to a 10% in 50 year motion. In other parts of the country, there is no simple correlation of the 2/3 factor with the probability or return period of earthquakes. One of the major reasons that the IBC applies this 2/3 factor is to ensure a minimum level of seismic design for buildings in lower seismicity parts of the United States. In California, the 2/3 factor results in a design level that is in general considered to be cost effective for assuring life safety goals for buildings. Outside of high seismicity parts of California, the 2/3 factor recognizes that ductile styles of building construction usually have a 1.5 factor of safety, and thus there should be reasonable life safety assurance for the 2,475 year earthquake, albeit the cost-effectiveness test may not be as well met. However, these Guidelines do not adopt this "2/3" factor in consideration that:

- Unlike the IBC, these Guidelines are for the design of pipelines. For many styles of pipeline design, there is no guarantee that there is an equivalent built-in factor of safety of 1.5. For non-ductile failure modes (such as pull out of joints for segmented pipelines, or wrinkling of continuous welded steel pipelines), the factor of safety for the design may be much less than 1.5.

- Unlike the IBC, the failure of a single pipeline in an entire water system pipeline network may not result in overall significant outage times, loss of fire service or economic disruptions to a community.

- Unlike the IBC, the Guidelines require use of the 975 and 2,475 year motion to provide the desired margin for the most important pipelines. The Guidelines do not use an importance factor "I". Therefore, the "2/3" factor should not be used to reduce the 2,475 year motion.

C4.4.2.1 Alignment Ground Class Definitions

To establish the site specific ground classification, the following procedure may be used; or the site classification can be specified by a suitable engineer / engineering geologist / geotechnical engineer professional. The notations below apply to the upper 100 feet of the site profile. Profiles containing distinctly different soil layers should be subdivided into those layers designated by a number that ranges from 1 at the top to n at the bottom where there are a total of n distinct layers in the upper 100 feet. The symbol, i, then refers to any one of the layers between 1 and n.

$$\overline{V}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}} \qquad \text{[Eq C4-1]}$$

where

$$\sum_{i=1}^{n} d_i = 100 \quad feet$$

v_{si} = the shear wave velocity in feet per second

d_i = the thickness of any layer between 0 and 100 feet

N_i is the Standard Penetration Resistance (ASTM D 1586-84) not to exceed 100 blows per foot as directly measured in the field without corrections.

$$\overline{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \qquad \text{[Eq C4-2]}$$

$$\overline{N_{ch}} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N_i}} \qquad \text{[Eq C4-3]}$$

where

$$\sum_{i=1}^{m} d_i = d_s$$

Use only d_i and N_i for cohesionless soils.

$d_s =$ the total thickness of cohesionless soil layers in teh top 100 feet

$$\overline{S_u} = \frac{d_c}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}} \qquad \text{[Eq C4-4]}$$

where

$s_{ui} =$ the undrained shear strength in psf, not to exceed 5,000 psf, ASTM D 2166-91 or D 2850-87.

$$\sum_{i=1}^{k} d_i = d_c$$

$d_c =$ the total thickness (100-d_s) of cohesive soil layers in the top 100 feet

$PI =$ the plasticity index (ASTM D 4318)

$w =$ the moisture content in percent (ASTM D 2216)

Steps for classifying a site.

- Check for the presence of Site Class F. For pipes that are important and that traverse Site Class F, it is recommended that site specific ground motions be developed, especially if using the FEM. Preliminary evaluations of such pipelines could be done using the simplified methods in Table 4-3, but with increased uncertainty.

- Check for the existence of a total thickness of soft clay > 10 feet where a soft clay layer is defined by $\overline{s_u} < 500$ psf, $w > 40$ percent, and $PI > 20$. If these criteria are satisfied, classify the site as Site Class E.

- Categorize the site using one of the following three methods with $\overline{v_s}$, $\overline{N}$, and $\overline{s_u}$, computed in all cases as specified.

  - $\overline{v_s}$ for the top 100 feet ($\overline{v_s}$ method)

- $\overline{N}$ for the top 100 feet ($\overline{N}$ method)
- $\overline{N_{ch}}$, for cohesionless soil layer ($PI < 20$) in the top 100 feet and average $\overline{s_u}$ for cohesive layers ($PI > 20$) in the top 100 feet

C4.4.2.3 Near-source factors

Pipelines located within 15 km of the seismic source can be subjected to near-source seismic shaking, resulting in significantly larger ground motion parameters and ground strain than pipes located further away from the source.

Near source factors consider that:

- In the direction of fault rupture, ground motions are know to be greater.
- When oriented normal to the fault plane, ground motions are known to be larger than those oriented parallel to the fault plane.
- At sites on the hanging wall of non-vertical faults, ground motions are larger than on the foot wall.

The USGS PSHA implicitly accounts for directivity or hanging wall as such variations in ground motions are included in the standard error terms that are part of the PSHA. This is not to say that at some location along the length of a pipeline that there might not be some exceedance in the ground motion. However, prudence suggests that it is not cost effective to design pipelines for the maximum possible ground motion that can theoretically occur at any location along the pipeline alignment as this will lead to cost-ineffective solutions.

Should the user wish to design Function Class II, III or IV pipelines using DSHA, the effects of fault normal, fault parallel, directivity, hanging wall or other seismologic effects can be included. Including all such effects, the ground motion used for design of the pipeline should not exceed the 50th, 67th or 84th non-exceedance percentile motions for Function Class II, III or IV pipelines, respectively.

C4.4.2.5 Design Response Spectra

The NEHRP 2003 and ASCE 7-2005 introduce a new parameter, T_L, that changes the long period portion of the spectrum.

C4.4.2.6 Fault Movement

A separate PSHA may be applied to fault displacement to determine probabilities that various displacement amplitudes will be exceeded. This would provide results of uniform confidence consistent with the ground motion parameters utilized for these Guidelines. The ground motion PSHA daggregation identifies seismic parameters needed to perform uniform evaluations. However, use of these parameters alone (e.g., M for fault displacement) does not provide adequate information for estimating fault displacements at the same uniform confidence level. There remains a certain probability that the deformation may be exceeded. The current technology is not at a state to provide consistent recommendations for uniform confidence of surface fault displacement. Therefore, estimates are presented in Table 4-5 to approximate the confidence level recommended in Table 3-2.

The recommendations for design of fault rupture displacement presented herein are consistent with the concept that faults generally rupture with a limited range of characteristic magnitudes and within the range of characteristic earthquakes, there remains some uncertainty of the magnitude and resulting surface fault displacement at any location along the fault trace. Thus, it would be inappropriate to reduce the design active fault displacement based on a fault rupture recurrence interval longer than the design return period identified in Table 3-2, as this would result in the design for a surface fault rupture corresponding to a *M* less than the characteristic magnitude (i.e., a surface rupture that will certainly be exceeded).

C4.4.2.7 Liquefaction Assessment

The assessment of liquefaction triggering is best performed using field data such as Standard Penetration Tests (SPT), Cone Penetration Tests (CPT), shear wave velocity, or other appropriate means. Youd and Idriss (1997) present an overview of state-of-the practice techniques for assessing liquefaction triggering. A liquefaction triggering evaluation requires the understanding of soil properties, groundwater elevation, and earthquake hazard. The confidence level in for the liquefaction triggering assessment is dependent upon the level of knowledge in each of the evaluation parameters. The earthquake hazard parameters need in the evaluation (*PGA* and *M*) can be determined directly from the PSHA. The soil properties and groundwater elevation and their variation are recommended to be assessed to the same degree of confidence as the earthquake parameters. Methods assessing the probability of liquefaction actually triggering are provided by Juang et al (2002). The potential for liquefaction triggering is recommended to be assessed to the same confidence level for the pipe Function as recommended in Table 3-2.

The level of uncertainty in using data in Table 4-6 for liquefaction triggering is relatively high, unless in-situ soil data is obtained, and therefore requires a certain level of conservativeness in assessing overall liquefaction potential. A very important aspect in liquefaction evaluation is the understanding of groundwater level and its fluctuation. It is important to determine if soils that are unsaturated at the time of evaluation may become saturated at some later time.

C4.4.2.7 Liquefaction Induced Permanent Ground Movement

The PGD_L from Equation 4-9 used with the factors presented in Table 4-8 provides the design movement for each pipe function as recommended in Table 3-2. The Bardet et al. (2002) multiple linear regression (MLR) is recommended for use with these Guidelines because they are simpler to use than other relations and the analysis allows for determination of a confidence level, which is not possible with other permanent ground deformation evaluations (e.g., Youd et al., 2002). If grain size distributions in the soil are known, the Youd et al. (2002) MLR results are recommended.

C4.5 Fault Offset

Potentially active faults (activity within 11,000[1] years to 1,900,000 years ago) generally need not be considered for design purposes. However, the owner may wish to consider sympathetic movements on potentially active faults on the order of 10% of the movement of a nearby active fault, for Function Class IV pipelines. This movement might occur in conjunction (of within a few days thereof) of a major offset on a nearby active fault.

Fault offset can be estimated using Equations [4-8 through 4-10]. Fault offset can also be estimated using historical evidence, paleoseismic evidence and/or slip rate calculations.

A more refined approach to define the design-basis fault offset than using Table 4-5 is to consider the uncertainty in the magnitude of the earthquake as well as the uncertainty in the amount of offset given a particular magnitude earthquake occurs. Figures C4-1 and C4-2 illustrate this process. In Figure C4-1, the range of uncertainty for a particular segment of a fault (northern Calaveras) is shown. the relative probability of occurrence of different magnitudes can be derived from the USGS web site as part of their ground shaking models, or by using expert opinion from knowledgeable seismologists. As can be seen, there is some disagreement between scientists about what the maximum M can be, ranging from M 6.2 to M 7.2. By suitably integrating the magnitude relationship in Figure C4-1 with the displacement model (such as Equation 4-8), allowing for uncertainty bands in that model, one can develop the resulting curves shown in Figure C4-2.

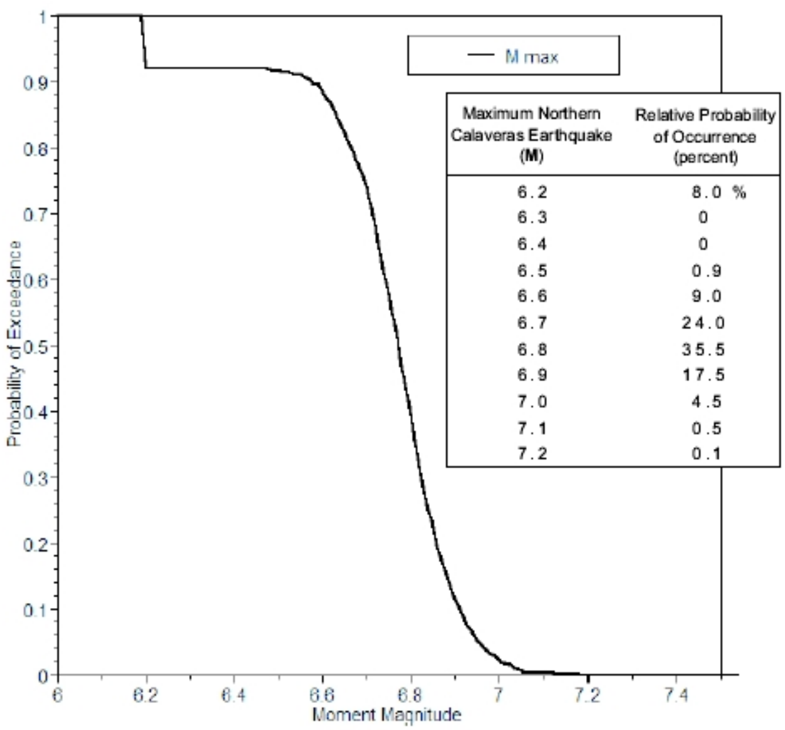
The final step of selecting the design offset displacement should consider the desired target reliability for the pipeline, which factors in the acceptable strain limit set for the pipeline. If one sets the acceptable strain for the pipe to be 5% in tension (assuming no compression in the pipe) and that at this level of strain, there is about 15% chance of failure, and if one sets the design motion at 84% not-to exceed level, then the combined reliability of the pipeline (assuming no other pipeline hazards) would be about 97%, given the maximum earthquake. This seems to be a reasonable, achievable design goal for keeping an essential water transmission pipeline in service.

For Function Class III pipes that are designed for fault offset, the fault offset Design Movement PGD is taken from 0.75*Dmax curve (Figure C4-2) at 50% chance of exceedance. (Dmax refers to the maximum offset that would be measured at any location along the length of the surface rupture.)

[1] California uses 11,000 years for Holocene; the rest of the world uses 10,000 years. For cost effective design, it is not necessary to design water pipes for fault offset across faults that have not moved in the last ten thousand years or so.

For Function Class IV pipes that are designed for fault offset, the fault offset Design Movement PGD is taken from 0.75*Dmax curve (Figure C4-2) at 16% chance of exceedance.

Depending upon the dispersion in the maximum magnitude M, the resulting Design Movement PGD using this approach may be higher or lower than that in Table 4-5. If both the approaches in Table 4-5 and Figures C4-1 and C4-2 are used, then the Design Movement PGD should be based on Figures C4-1 and C4-2.



Maximum Northern Calaveras Earthquake (M)	Relative Probability of Occurrence (percent)
6.2	8.0 %
6.3	0
6.4	0
6.5	0.9
6.6	9.0
6.7	24.0
6.8	35.5
6.9	17.5
7.0	4.5
7.1	0.5
7.2	0.1

Figure C4-1. Probability of Exceedance of Magnitude M

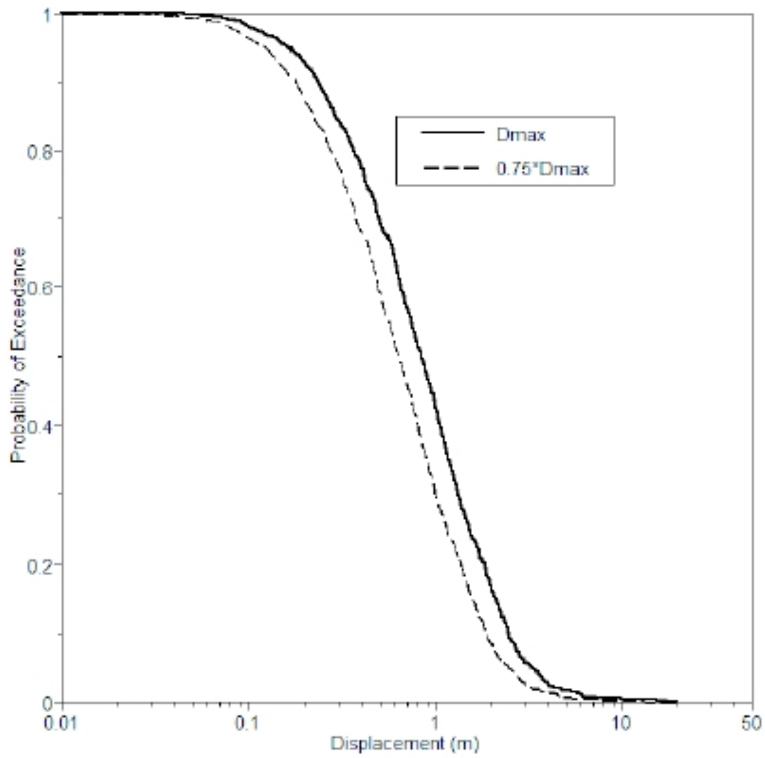


Figure C4-2. Probability of Exceedance of Fault Offset Displacement

A simple design approach for butt-welded continuous steel pipes is to adopt the recommendations in Table 4-6 or Figures C4-1 and C4-2, and assume that there is a knife edge fault offset anywhere in Zone A (Figure 4-5), or at the location which produces the highest forces on adjacent pipeline and appurtenances, and then extend the Zone A design through Zone B. However, for fault zones with multiple traces, or for design cases using chained-segmented pipes, such a simplification might result in too few chained joints, and consideration of the full variation of fault offset possibilities is required.

For the design of new oil and gas pipelines, a somewhat more conservative approach is often adopted to obtain the design fault offset displacement. In these cases, for faults that are determined to be active, one assumes offset of the entire fault (in other words, the smaller mean or modal magnitudes from Figure 4-3 are ignored). Given the rupture of the entire fault, the Design Movement PGD is based on the mean average fault displacement (AD) unless rupture of the pipe would result in extreme consequences, in which case use mean maximum fault displacement (MD) (e.g. gas release in densely populated area, extremely adverse political consequences such as oil release into an environmentally sensitive area).

When using Table 4-5, it is intended that Function Class IV pipelines will be designed to accommodate about the 84th percentile not-to-exceed fault offset displacement of the next large earthquake on the fault, given uncertainties in M and displacement given M. If 2.3*AD results in a fault offset that is higher than this limit, it can be scaled down to this limit.

C4.6 Liquefaction

Liquefaction is a phenomenon that occurs in loose, saturated, granular soils when subjected to long duration, strong ground shaking. Silts and sands tend to compact and settle under such conditions. If these soils are saturated as they compact and settle, they displace pore water, which is forced upwards. This increased pore water pressure causes two effects. First, it creates a quick condition in which the bearing pressure of the soils is temporarily reduced. Second, if the generated pressures become large enough, material can actually be ejected from the ground to form characteristic sand boils on the surface. This displaced material in turn results in further settlement of the site.

Lateral spreading is a phenomenon which can accompany liquefaction. On many sites, the layers of liquefiable materials are located some distance below the ground surface. If the site has significant slope, or is adjacent to an open cut, such as a depressed stream or road bed, liquefaction can cause the surficial soils to flow downslope or towards the cut. Lateral spreading can be highly disruptive of buried structures and pipelines, as well as structures supported on the site.

When applying PGD estimates for liquefaction, it is proposed that Function Class II pipelines in locations where liquefaction will occur in a 2,475 year event, but not in a 475 year event, be designed for 1/3 the PGD associated with the 2,475 year event. Where liquefaction will occur in a 2,475 year event, but not in a 975 year event, Function III pipelines should be designed for 2/3 the PGD associated with the 2,475 year event.

One way to evaluate the liquefaction hazard along a specific pipeline right-of-way is to perform site-specific liquefaction analyses. Such an approach would be undertaken with the use of Eq. 4-11. Equation 4-11 assumes that liquefaction occurs (scenario based) at a particular location. In many instances, the pipeline engineer will not have available the parameters needed to use Eq. 4-11 (W, S and T_{15}); especially for Function Class II pipelines.

For Function Class II pipelines, even if all parameters are known, Eq. 4-11 might lead to non-cost-effective conservatism. For Function Class II pipelines, a probabilistic approach might be more suitable. This can be done using suitably-prepared regional liquefaction hazard maps. Examples for three such maps are given in Figures C4-3, C4-4 and C4-5.

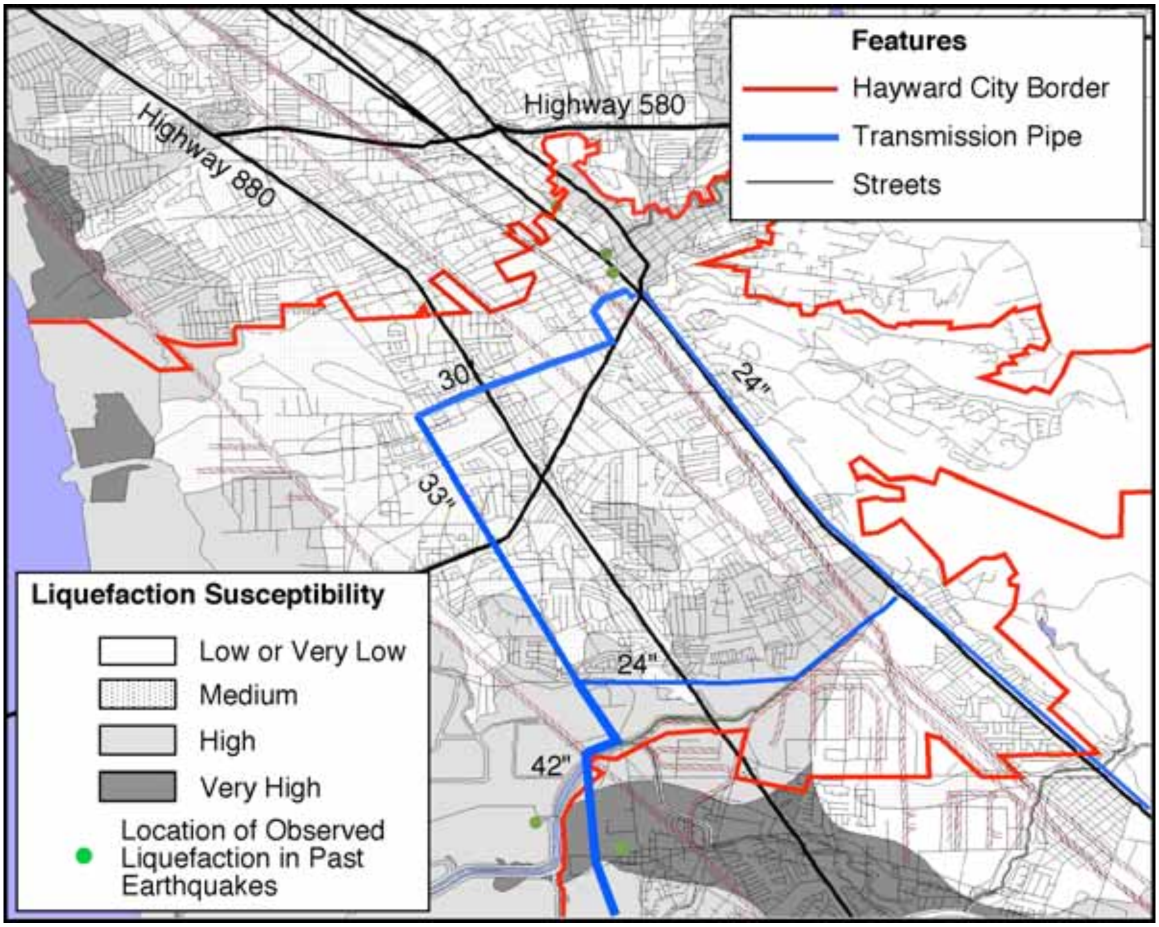


Figure C4-3. Regional Liquefaction Susceptibility Map –Hayward Area

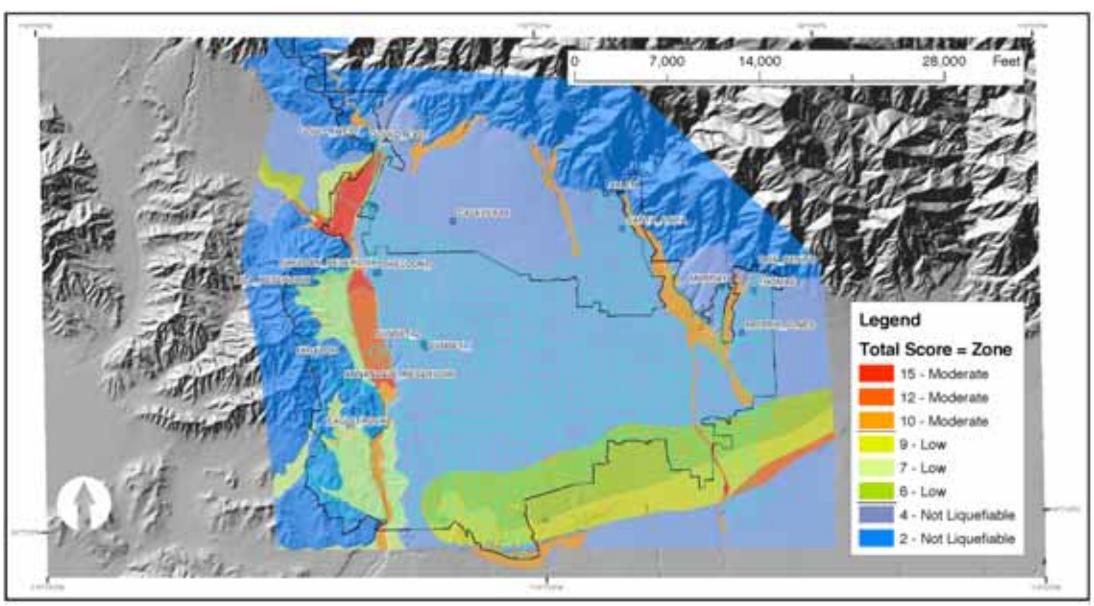


Figure C4-4. Regional Liquefaction Susceptibility Map – Pasadena Area

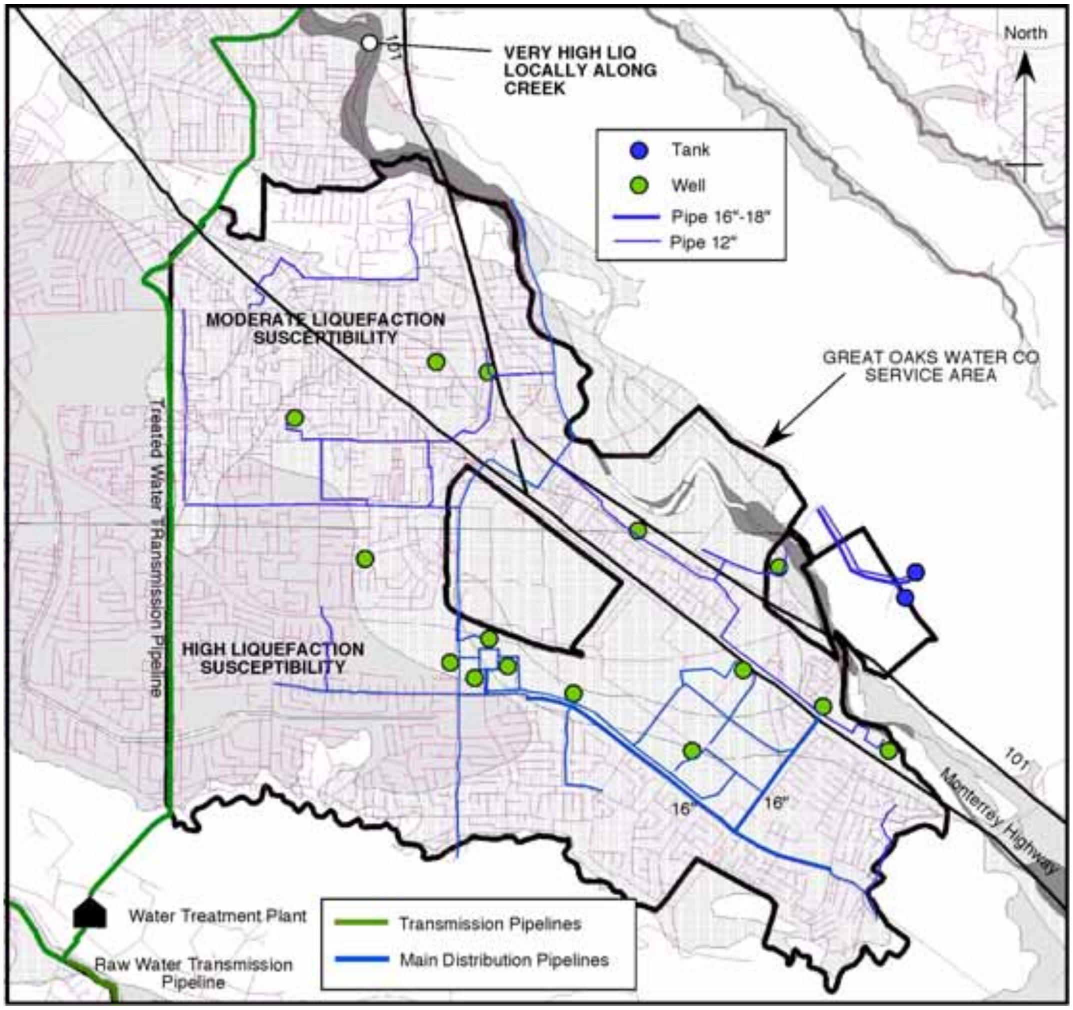


Figure C4-5. Regional Liquefaction Susceptibility Map – South San Jose Area

For many urbanized areas of the country, liquefaction susceptibility maps have already been prepared; see Power and Holzer (1996) for a detailed bibliography of available liquefaction maps. Ongoing and past consultant studies by firms such as Geomatrix, WLA, Woodward Clyde (now URS) and others have already produced similar maps for the following water utilities or regions:

- San Diego Water Department
- Santa Clara Valley Water District
- East Bay Municipal Utility District
- Greater Memphis Tennessee area
- Greater Salt Lake City area
- Greater Portland Oregon area
- Greater Seattle Washington Area
- Pasadena California

- And many others

Recent "seismic hazard zone" maps prepared by the CGS (formerly CDMG) for purposes of establishing liquefaction special study zones are in general not directly suitable for buried pipe design, in that the CGS liquefaction (and landslide) zones are not defined by the level of hazard, and do not verify that any hazard in fact exists at a particular location (CGS). While these maps could be used as a starting point in developing a liquefaction susceptibility map for pipeline design purposes, the CGS maps should not be used directly as part of the pipeline design approach outlined in these Guidelines.

C4.6.1 Simplified Method to Prepare a Regional Liquefaction Map

A regional liquefaction map should ideally link liquefaction susceptibility categories, such as "very high", "high", etc., with the fragility models used to forecast the level of PGD within these regions, as well as fragility models use to forecast the amount of pipe damage, given the PGD occurs.

The map and associated documentation should provide an estimate of the probability that a specific site will liquefy, and if it does, the amount of permanent ground deformation (PGD) at that site. The PGD can be either vertical (settlement) or lateral (lateral spread) or a combination of the two. If there is a combination, the vector sum value of PGD should be used for use for pipeline design.

Commentary Section C4.6.1 provides a way to establish PGDs given the development of liquefaction hazard maps. These procedures follow the methodology in HAZUS (1997) and have been benchmarked in conjunction with pipe fragility curves (ALA 2001) to give reasonable overall pipe damage patterns from past earthquakes. This process has been used in several large-scale loss estimates for water utilities and has been benchmarked with regards to actual observed pipe damage in past earthquakes. The HAZUS software is based on this model. However good the benchmarking has been done, it is recognized that the process validates only that the cumulative loss through forecast of PGA, PGD and damage due to PGD is verified; and the intermediate steps (PGA, PGD) are only benchmarked against limited empirical observations.

No consensus was achieved amongst the authors of these Guidelines (or the wider industry as a whole) regarding easy-and-inexpensive-to-use processes for estimating liquefaction-induced PGD and associated locations of the movement. The absence of an accepted, rigorous procedure for characterizing the spatial variability of liquefaction-induced PGD and the probability of its occurrence requires additional research and development, and is a prime topic for future investigation related to developing a design process for seismic resistant buried water pipelines.

With these important caveats in mind, a simplified approach is described in this commentary, as follows.

An assessment of liquefaction susceptibility may be performed using a regional seismic hazard map, similar to those illustrated in Figures C4-3 through C4-5. The assessment should be performed by qualified geotechnical engineers and geoscientists and should incorporate borehole information, water level data, and historical information about the effects of previous earthquakes where such information is available. Areas on the map should be identified and characterized according to liquefaction susceptibility following Table 4-6 and using procedures to deduce liquefaction susceptibility from SPT and CPT data, as described in Chapter 5.

From the map, zones designated as having “very high” and “high” susceptibility should be regarded as zones for potential strengthening and protective measures in pipelines either located in or planned for these areas. Using the advice of qualified geotechnical engineers and geoscientists, an estimate of the percentage of each zone designated with “very high susceptibility” and “high susceptibility” that would experience PGD exceeding 1 feet should be made for the 2475-year event. Restrained joints should be used in all new pipelines and pipeline replacements in each zone with “very high” and “high” susceptibility where more that 50% of the zone is predicted to experience PGD ≥ 1 feet. The highest priority for strengthening must be given to those pipelines so designated in the “very high susceptibility” zones.

The proposed use of regional liquefaction hazard maps acts as a screening process to identify a limited number of pipelines at highest risk from liquefaction effects. The approach promotes seismic protection, and will often result in some measure of strengthening for areas of the US most seriously affected by low frequency, high consequence events. The extent and degree of improvement, however, is constrained within limited geographical bounds to reduce cost and limit the time required for detailed planning and assessment by owner/operator staff.

For practical purposes, most regularly designed (Function I) buried pipelines will sustain damage at PGDs much over a foot; hence extreme accuracy in calculation of the PGD parameter is not essential in these cases.

To proceed with a simplified first order evaluation of a water pipeline, the liquefaction hazard at any location can be calculated in the following steps:

1. For a scenario earthquake (see Section 4.4), calculate the level of shaking (PGA, g) at the particular location of the pipeline to be designed.
2. Establish the geologic unit for the near surface environment at the pipeline location. Table 4-6, after Youd and Perkins (1978) provides a liquefaction susceptibility description for several types of sedimentary deposits.
3. Given the PGA (a, in g), geologic unit and liquefaction susceptibility description, the estimated groundwater depth and the magnitude of the earthquake (using magnitude as a proxy for earthquake duration), calculate the probability that liquefaction occurs at the location, noting that increased magnitude leads to increased chance of liquefaction. A simplified method is provided in equation [C4-

6]. The validity of equation [C4-6] is highly influenced by actual localized soil conditions and groundwater depth, and should not be used without some form of validation for the local soil conditions.

$$P[\text{liquefaction}] = \frac{P[\text{liquefaction}|PGA = a]}{K_m K_w} P_{ml} \qquad \text{[Eq C4-6]}$$

where

$P[\text{liquefaction}|PGA = a] =$ probability of liquefaction given a specified PGA (Table C4 - 3)
$K_m =$ the moment magnitude correction factor, equation (C4 - 7)
$K_w =$ the ground water correction factor, equation (C4 - 8)
$P_{ml} =$ the proportion of the map unit susceptible to liquefaction (Table C4 - 4)

$$K_m = 0.0027M^3 - 0.0267M^2 - 0.2055M + 2.9188 \qquad \text{[Eq C4-7]}$$

$$K_w = 0.022d_w + 0.93 \qquad \text{[Eq C4-8]}$$

Note that the liquefaction probability model in equation C4-6 incorporates the same measure of uncertainty as is used to establish PGA=a. In reality, the three other parameters in equation C4-6 (K_m, K_w and P_{ml}) are also uncertain; however, the state of the practice in liquefaction modeling usually does not specify uncertainties for these parameters. One possible approach to this limitation is to increase the ground motion attenuation beta (standard deviation of ln a, say from 0.4 to 0.5) for purposes of using this equation; it is left to the expert geotechnical hazard practitioner to quantify this for any specific project.

Liquefaction Susceptibility (From Map or Table 4-7)	$P[\text{liquefaction}\|PGA = a]$
Very High	9.09 a - 0.82
High	7.67 a - 0.92
Moderate	6.67 a - 1.00
Low	5.57 a - 1.18
Very Low	4.16 a - 1.08
None	0.00

Table C4-3. Conditional Probability Relationship for Liquefaction Susceptibility Categories [after Liao et al 1988]

The model in Table C4-3 is based on a moment magnitude 7.5 earthquake and an assumed groundwater depth of five feet. For example, if a = 0.20g and the liquefaction susceptibility description is "High", then the probability of liquefaction in a map unit is 7.67 * 0.20 - 0.92 = 0.614 (61.4%). If the value a=0.20g was at the median level of motion, then the median chance of liquefaction is 61.4% (using equation C4-6). The equations in Table C4-3 are bounded by 0.0 and 1.0. Equations [C4-7] and [C4-8] are used to adjust the model for different moment magnitudes (Seed and Idriss, 1982) and groundwater depths (d_w in feet).

Not all soils within a map unit will have the same liquefaction susceptibility. For liquefaction maps based on wide area geology maps, there will usually be considerable variation of soils within a single mapped soil unit. To approximately account for this spatial variability within a mapped soil unit, Table C4-4 is used. Note that Tables C4-3, C4-4 and C4-5 are linked, and changes in one table could influence the other tables.

Liquefaction Susceptibility (From Map or Table 4-7)	Proportion of mapped unit, P_{ml}
Very High	0.25
High	0.20
Moderate	0.10
Low	0.05
Very Low	0.02
None	0.00

Table C4-4. Proportion of Mapped Unit Susceptible to Liquefaction

4. Given that the site liquefies, calculate the maximum permanent ground deformation (settlement). Table C4-5 provides a chart to estimate settlements.

Settlement Range (inches)	Probability Range for Soil Susceptibility		
	Very High	High	Low to Moderate
≤ 1	0 %	0 %	35 %
1 to 3	5 %	55 %	60 %
3 to 6	25 %	30 %	4 %
6 to 12	50 %	12 %	1 %
> 12	20 %	3 %	0 %

Table C4-5. Probable Ground Surface Settlements, Given Liquefaction Occurs

5. If the site is located adjacent to an open cut (often the case when near a body of water), and the site liquefies, there is a chance that it will also displace sideways (lateral spreading). Equation [C4-9] can be used to estimate the amount of lateral movement.

$$E\left[PGD\right] = K_d * E\left[PGD \middle| \left(PGA/PGA(t)\right) = x\right] \qquad \text{[Eq C4-9]}$$

where

$$K_d = 0.0086M^3 - 0.0914M^2 + 0.4698M - 0.9835 \qquad \text{[Eq C4-10]}$$

$PGA/PGA(t)$, = x	PGD (inches)
1 to 2	12x - 12
2 to 3	18x - 24
3 to 4	70x - 180

Table C4-6. Lateral Spreading Displacement Relationship [after Youd and Perkins, 1978, Sadigh et al 1986] (for x>4, use x=4)

For example, assume a site with very high liquefaction susceptibility, and PGA = a = 0.36 g, a moment magnitude 7 event, and liquefaction does occur, and site is located adjacent to an open cut or is suitably sloped. Then, the expected lateral PGD would be 78 inches. (PGA(t) = 0.09g from Table C4-7. PGA/PGA(t) = 4. PGD = 70 (4) - 180 = 100 inches, from Table C4-6. K_d = 0.78.

Liquefaction Susceptibility (From Map or Table 4-7)	PGA(t)
Very High	0.09 g
High	0.12 g
Moderate	0.15 g
Low	0.21 g
Very Low	0.26 g
None	not applicable

Table C4-7. Threshold Ground Acceleration (PGA(t)) Corresponding to Zero Probability of Liquefaction

The associated range of PGD is assumed to have a uniform probability distribution within bounds of one-half to two-times the displacement calculated using equation (C4-16). For the example given above, the lateral spread displacement would be described as a range between 39 inches to 156 inches.

Additional methods to estimate the effects of liquefaction are provided in the 1997 liquefaction workshop (Youd and Idriss, 1997).

C4.6.2 Buoyancy

Pipe damage to sewer pipes due to buoyancy has been commonly observed in a variety of earthquakes in Japan.

It is felt that practical engineering assessments of pipes can be made by considering the residual strength of the soil.

C4.6.3 Settlement

The simplified approach for settlements using Table C4-5 and equation [C4-6] will result in a range of possible settlements. A conservative design approach for new pipe installation should subsurface information not be available would be to adopt about the

80% non-exceedance level settlements in Table C4-5. For very high susceptible areas, this is 12 inches settlement. For high susceptible areas, this is 6 inches settlement. For moderate susceptible areas, this is 2 inches settlement.

This approach is conservative since the settlements are towards the upper bound, and ignore the fact that large areas within a susceptible zone will not settle at all. Multiplying these settlements (12/6/2 inches) by equation [C4-6] is a reasonable conservative approach when estimating overall system pipe damage estimates.

C4.6.4 Spatial Variation of Liquefaction PGDs

It is ultimately the task of the engineer to select a PGD pattern that reflects the spatial extent of the liquefaction zone, the topography, and the pipeline design approach in order to establish suitable spatial variations of PGD to be considered in pipeline design. A geosciences expert may help define the spatial variation of the PGD for liquefaction, landslide and fault offset for the particular situation at hand.

C4.6.5 Application of Regional Liquefaction Map

The design of water pipelines, especially buried water pipelines, can be largely controlled by the presence of soils subject to permanent ground deformations (PGDs). The PGDs could be from liquefaction, landslide or surface faulting, for example.

It has been the observation of several water utilities that most soils prone to liquefaction-induced PGDs are also highly corrosive. Even after many past earthquakes, it still remains somewhat unclear to what extent observed pipeline damage has been due to PGD, corrosion, or some combination of both. It is likely that there is a high correlation between the two processes.

C4.7 Landslide Assessment

The procedure to estimate PGD in the commentary is adopted from HAZUS (1997).

Landslide hazards encompass several distinct types of hazard. There are deep seated and rotational landslides; debris flows; and avalanche / rock falls. These different types of landslides can affect water pipelines in different ways:

- Buried pipelines, valves and vaults. Deep seated rotational and translational landslides pose a significant threat to causing damage to buried pipelines, valves and vaults. Most past efforts in estimating landslide-induced damage to water pipelines has been for deep seated landslides. Debris flows and avalanches are usually not credible threats to buried structures.

Section C4.7 discusses hazard models for deep seated landslide movements. These Guidelines do not present models for debris flows, rock falls or avalanches. If a particular water pipeline appears vulnerable to these types of landslides, then a site specific hazard model should be developed.

There are three basic steps in evaluating the deep seated landslide hazard:

- Develop a landslide susceptibility map.
- Estimate the chance of landslide given an earthquake.
- Given that a landslide occurs, estimate the amount and range of movement.

Landslide Maps. This effort should be performed by geologists familiar with the geology of the area. There are many ways to develop such maps, ranging from aerial photo interpretation to field investigation to borehole evaluations. The cost to develop these maps can be substantial, especially if there are no available maps.

For some areas, landslide susceptibility maps have already been prepared. For example, the USGS has issued a number of such maps (Nielson, 1975). Recent "seismic hazard zone" maps prepared by the CGS for purposes of establishing landslide special study zones are in general not directly suitable for loss estimation, in that the CGS landslide (and liquefaction) zones are not defined by the level of hazard, and not verified that any hazard in fact exists (ref. CGS); while these maps could be used as a starting point in a water pipeline design effort, these maps should not be used with the design procedures presented in these Guidelines. Site specific surveys and aerial photographs can be used for specific pipeline alignments.

Earthquake-induced landsliding of a hillside slope occurs when the static plus inertia forces within the slide mass cause the factor of safety to temporarily drop below 1.0. The value of the peak ground acceleration within the slide mass required to just cause the factor of safety to drop to 1.0 is denoted as the critical or yield acceleration, a_c. This value of acceleration is determined based on pseudo-static slope stability analyses and/or empirically based on observations of slope behavior during past earthquakes.

Deformations can be calculated using the approach originally developed by Newmark (1965). The sliding mass is assumed to be a rigid block. Downslope deformations occur during the time periods when the induced PGA within the slide mass, a_{is} exceeds the critical acceleration a_c. In general, the smaller the ratio below 1.0, of a_c to a_{is}, the greater is the number and duration of times when downslope movement occurs, and thus the greater is the total amount of downslope movement. The amount of downslope movement also depends on the duration or the number of cycles of ground shaking. Since duration and number of cycles increase with earthquake magnitude, deformation tends to increase with increasing magnitude for given values of a_c to a_{is}.

The landslide evaluation requires the characterization of the landslide susceptibility of the soil / geologic conditions of a region or subregion. Susceptibility is characterized by the geologic group, slope angle and critical acceleration. The acceleration required to initiate slope movement is a complex function of slope geology, steepness, groundwater conditions, type of landsliding and history of previous slope performance. At the present time, a generally accepted relationship or simplified methodology for estimating a_c has not been developed. The relationship proposed by Wilson and Keefer (1985) is suggested, shown in Figure C4-6. Landslide susceptibility is measured on a scale of I to X, with I being the least susceptible. The site condition is identified using three geologic groups and groundwater level. The description for each geologic group and its associated susceptibility is given in Table C4-8. The groundwater condition is divided into either dry condition (groundwater below level of the sliding) or wet condition (groundwater level at ground surface). The critical acceleration is then estimated for the respective geologic and groundwater conditions and the slope angle. To avoid calculating the occurrence of landsliding for very low or zero slope angles and critical accelerations, lower bounds for slope angles and critical accelerations are established. These bounds are shown in Table C4-9.

Geologic Group		Slope Angle, Degrees					
		0-10	10-15	15-20	20-30	30-40	>40
(a) Dry (groundwater below level of sliding)							
A	Strongly cemented rocks (crystalline rocks and well-cemented sandstone, (c'=300 psf, ϕ=35˚)	None	None	I	II	IV	VI
B	Weakly cemented rocks (sandy soils and poorly-cemented sandstone, (c'=0 psf, ϕ=35˚)	None	III	IV	V	VI	VII
C	Argillaceous rocks (shales, clayey soil, existing landslides, poorly compacted fills), (c'=0 psf, ϕ=20˚)	V	VI	VII	IX	IX	IX
(b) Wet (groundwater level at ground surface)							
A	Strongly cemented rocks (crystalline rocks and well-cemented sandstone, (c'=300 psf, ϕ=35˚)	None	III	VI	VII	VIII	VIII
B	Weakly cemented rocks (sandy soils and poorly-cemented sandstone, (c'=0 psf, ϕ=35˚)	V	VIII	IX	IX	IX	X
C	Argillaceous rocks (shales, clayey soil, existing landslides, poorly compacted fills), (c'=0 psf, ϕ=20˚)	VII	IX	X	X	X	X

Table C4-8. Landslide Susceptibility of Geologic Groups

	Dry Conditions	Wet Conditions	Dry Conditions	Wet Conditions
A	15	10	0.20	0.15
B	10	5	0.15	0.10
C	5	3	0.10	0.05

Table C4-9. Lower Bounds for Slope Angles and Critical Accelerations for Landsliding Susceptibility

The relationships in Figure C4-6 are conservative and represent the most landslide-susceptible geologic types likely to be found in the geologic group. Thus, in using this relationship, further consideration must be given to evaluating the probability of slope failure, using Tables C4-10 and C4-11.

Table C4-10 provides landslide susceptibilities defined as a function of critical acceleration.

Using the relationship in Figure C4-6 and the lower bound values in Table C4-9, the susceptibility categories are assigned as a function of geologic group, groundwater conditions and slope angle in Table C4-8.

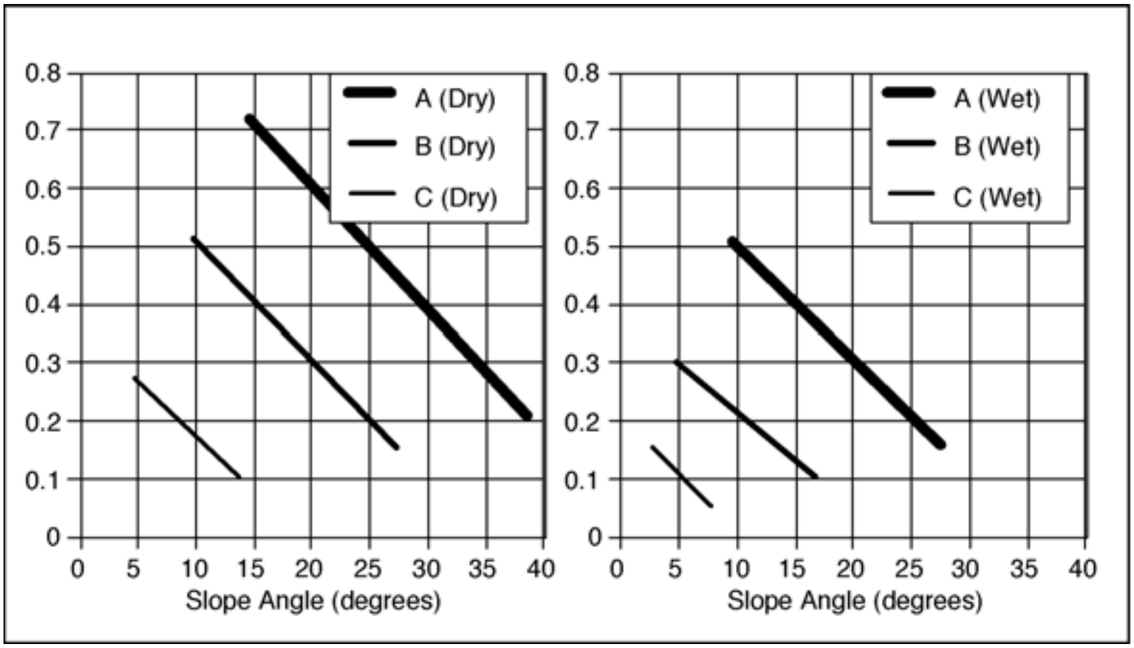


Figure C4-6. Critical Acceleration as a Function of Geologic Group and Slope Angle

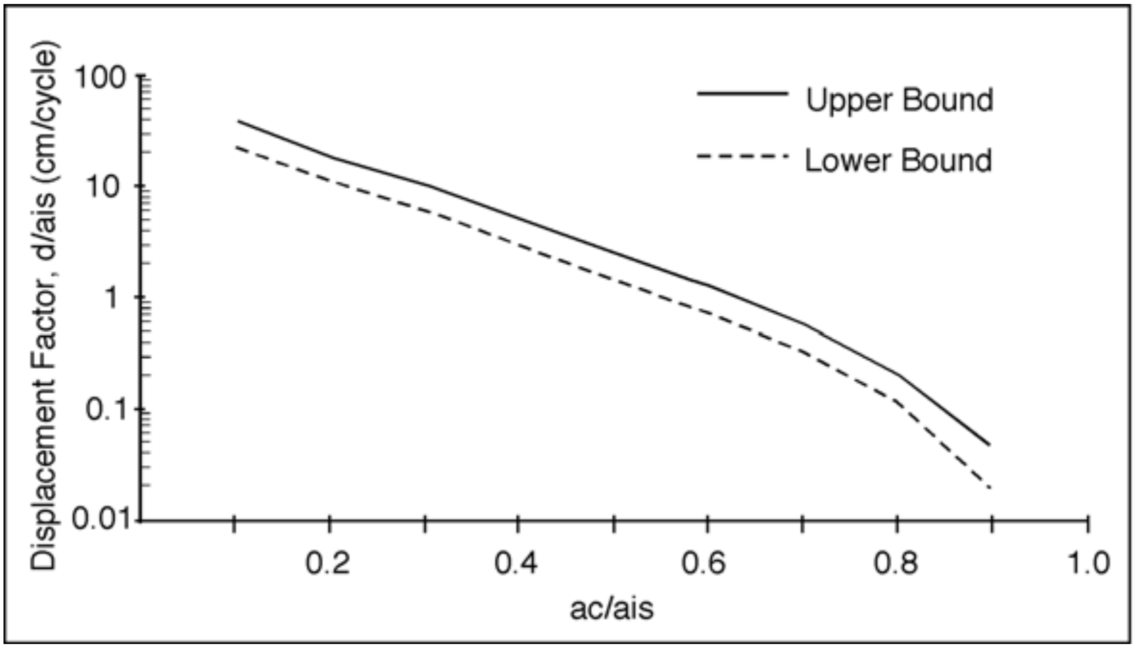


Figure C4-7. Relationship Between Displacement Factor and Ratio of Critical Acceleration and Induced Acceleration

Because of the conservative nature of Figure C4-6, an adjustment must be made to estimate the percentage of a landslide susceptibility category that is expected to be susceptible to landslide. Based on Wieczorek and others (1985), this percentage is estimated using the ratios in Table C4-11, which are presented as a ratio (0.01 = 1%). Thus, at any given location, landsliding either occurs or does not occur within a susceptible deposit depending on whether the peak induced PGA a_{is} exceeds the critical acceleration a_c.

For locations which do slide, the amount of PGD can be estimated using equation[C4 - 11]. Note that the uncertainty description in equation [C4-11] is governed by the uncertainty in the local induced ground acceleration, aig; however, it is clear from the formulation that there should also be some uncertainty for the other factors in the model; this could be roughly accounted for by increasing the ground motion uncertainty parameter to 0.5 or so; or by having a competent geotechnical engineer provide a site specific description of the uncertainties involved. It is beyond the scope of these Guidelines to assess this pipeline design process.

Susceptibility Category	None	I	II	III	IV	V	VI	VII	VIII	IX	X
Map Area	None	0.60	0.50	0.40	0.35	0.30	0.25	0.20	0.15	010	0.05

Table C4-10. Critical Accelerations (a_c) for Susceptible Categories

Susceptibility Category	None	I	II	III	IV	V	VI	VII	VIII	IX	X
Map Area	0.00	0.01	0.02	0.03	0.05	0.08	0.10	0.15	0.20	0.25	0.30

Table C4-11. Percentage of Map Area with Landslide Susceptible Deposit

$$E\left[PGD\right] = E\left[d / a_{is}\right] a_{is} n$$

where

$E\left[d / a_{is}\right] =$ the expected displacement per cycle, Figure C4 - 7 [Eq C4-11]

$a_{is} =$ the induced acceleration (in g)

$n =$ the expected number of cycles, equation [C4 -12].

The relationship between the number of cycles and moment magnitude is estimated using equation [C4-12], which is based on Seed and Idriss (1982).

$$n = 0.3419 M^3 - 5.5214 M^2 + 33.6154 M - 70.7692 \quad \text{[Eq C4-12]}$$

For relatively shallow and laterally small landslides, a_{is} is not significantly different from the induced PGA at the surface of the slide, a_i. For deep and large slide masses, a_{is} is less than a_i. For many applications, it may be reasonable to assume $a_{is} = a_i$. However, soil column deamplification and topographic amplification effects may be important in some cases. The uncertainty in any estimated landslide PGD is governed by the uncertainty in the local induced ground acceleration, and for other factors in the model; this could be roughly accounted for by using a suitable ground motion uncertainty parameter (perhaps 0.5 or so); or by having a competent geotechnical engineer provide a site specific description of the uncertainties involved. It is beyond the scope of this document to assess this uncertainty, other than to note that this value may be important in terms of the overall water pipeline design process.

C4.8 Ground Motion Parameters in Other Codes

The maps and procedures listed in the Guidelines to establish site-specific PGA and PGV values rely on country-wide maps and simplistic conversion tables to consider site-specific soil conditions.

2003 International Building Code

- Ground motion parameters based on 2% chance of exceedance in 50 years (=2,475 year recurrence interval).
- 5% damped response spectra are developed from the spectral acceleration at short periods (S_s) [determined at 0.2 second period] and at 1 second period (S_1). S_s and S_1 are determined from maps plotted for these parameters for all US states and territories for B/C rock sites.
- The site specific design parameters at short periods and 1 second period S_{MS} and S_{M1}, respectively, are determined from:
    - o $S_{MS} = F_a S_s$
    - o $S_{M1} = F_v S_s$
    - o F_a and F_v are site coefficients that define the spectral shape as a function of site conditions that differ from B rock sites.

- The design spectral response is determined from:
    - $S_{DS} = (2/3)S_{MS}$
    - $S_{D1} = (2/3)S_{M1}$
    - Note: These Guidelines do not provide this 2/3 factor. The 2/3 factor should not be used for the seismic design of water pipelines.
- The peak ground acceleration (zero period acceleration) is determined from $0.4S_{DS}$
- This recurrence interval chosen because the common 475 year recurrence interval used for West Coast seismic design in the UBC is considered to provide such low level ground motions for Midwest and Eastern regions of the United States as to result in seismic design that would not provide any real safety should a large earthquake (2% in 50 year) occur. Therefore the IBC adopted the 2,475 year return period and scaled it down to be similar *(with a fudge factor of 2/3 which rarely works accurately*) to the 475 year return period in high-seismic California. This effectively normalizes ground motion parameters to be ***inconsistent*** with regards to risk across the entire USA.

ASCE 7.02

- Same as IBC 2000.

1997 NEHRP Provisions

- Same as IBC 2000. IBC used same methods as presented by 1997 NEHRP.

UBC 1997

- Ground motion parameters based on 10% chance of exceedance in 50 years. 475 year recurrence interval.
- Site seismic hazard characteristics are established based on the seismic zone, site proximity to active seismic sources, site soil profile characteristics, and the facility importance.
- The seismic zone factor Z is determined from a map identifying regions of different shaking hazard for zones 1, 2a, 2b, 3, or 4. Z = 0.075 to 0.4.
- For Zone 4, each site is assigned a near source factor N_a based on the seismic source type. These near source factors are eliminated in most subsequent codes that are based on PSHA.
- Seismic coefficients C_a and C_v are assigned for each site based on the seismic zone and soil profile.
- Peak ground acceleration represented by C_a.
- Code specifies method for generating response spectra.
- Comparison with IBC:
    - Spectral shapes are developed the same.
    - $C_a = 0.4S_{DS}$
    - $C_v = S_{D1}$

JWWA

- Design for two different magnitudes of intensity
    - Strong Motion Level 1, L1, has a return probability of once or twice in the service life of the facility
        - Similar to standard motions for civil design.
- Strong Motion Level 2, L2, has a smaller probability than L1 and is greater in magnitude.
    - Motion generated in areas with faults or large scale plate boundaries bordering inland areas
    - Design basis is the 1995 Hyogoken Nanbu (Kobe) Earthquake.
    - If fault or plate boundary cannot be clearly defined then must design for L2.
        - See JWWA pages 16 to 32 for descriptions of ground motion parameter evaluations. Pages 28 to 32 cover peak ground motions and site natural periods.

C5.0 Subsurface Investigations

Table 5-1 provides guidance as to the type of information that is recommended for general and seismic pipeline design.

We have ranked the subsurface information to be collected in accordance with the pipe Function Class.

For Function Class II, we rely mostly on regional geologic information. With this information, plus the probabilistic PGD models in the Commentary, a rational approach can be taken to seismically design most distribution pipelines.

For Function Class III and IV, we suggest subsurface investigations. If a geoscience expert with knowledge of local soil conditions suggests that there are no liquefaction, landslide or faulting conditions along the pipeline alignment, then the subsurface program can be pared down to the minimum needed to provide the pipeline contractor with sufficient information to price the installation effort. The subsurface information in Table 5-1 would be useful at locations known (or suspected) prone to fault offset, lateral spread, landslide or substantial settlement.

C6.0 General Pipeline Design Approach

It is beyond the scope of these Guidelines to provide a complete treatment of the non-seismic design of buried water pipelines. Instead, we provide outlines of some of the main loading parameters that are commonly considered in non-seismic design. Moser (2001) provides a 600+ age book on the design of buried pipe. However, Moser (2001) only casually mentions that earthquake loading.

These Guidelines make no suggestion of how to combine seismic load cases with other load cases. Generally, the seismic load case leads to stresses in the pipe along the longitudinal axis of the pipe, and the most other load cases lead to stresses in the hoop (pressure) or through wall (external soil load) directions. Thermal loads are usually self-relieving, so need not usually be combined with seismic loads. Hydrostatic thrust and hydrodynamic thrust loads should be considered in conjunction with seismic loads.

For purposes of these Guidelines, seismic loads can be combined with other loads, where applicable, using unit load factors.

C6.6 Fluid Transients

Throughout these Guidelines, we make little mention of the effects of water within the pipeline on overall pipeline response. For buried pipes, this seems to be mostly true if considering just the effects of filled-pipe-soil interaction. However, there is continuing debate as to whether the forces due to pressure in the pipe are somehow increased during the earthquake, in part due to surge transients.

For above ground pipes, it is required to always include the mass of the water within the pipe as part of overall inertial loading for transverse and vertical loading. If the pipe bends are spaced closer than about 100 pipe diameters, it is rational to include the entire mass of water in the longitudinal as part of the dynamic analyses, when forecasting forces on adjacent bends in the pipe.

For above ground pipes that are straight for very long distances, such as many thousands of feet, it is too conservative to apply the entire mass of water as a constant inertial load to the bends at the ends of the straight run. As the pipe accelerates along the straight length, the bend at the end of the straight run will impose some dynamic impulses to the water, akin, in a way, to a valve closing transient, albeit with much shorter application time. It would be too conservative to apply this imposed loading to the water over the entire length of long straight pipe.

C7.0 Analytical Models

Nothing in these Guidelines should be taken as a recommendation to install one kind of pipe over another, as long as a rational analysis can show that the installed pipe will meet the intended performance.

C7.1 Three Models, and When to Use Them

In some other reports, the Finite Element Method is sometimes called a "dynamic analysis method". But for buried pipelines there are rarely any pipe mass or velocity terms important to pipeline response, as the pipe usually moves more-or-less with the soil and the pipe itself rarely has any dynamic amplification; thus we avoid the term "dynamic analysis method" in this report. This statement does not apply to hydrodynamic forces of the water within the pipe.

In most cases when using the Finite Element Method, it will be sufficient to just apply PGDs to the pipeline. PGA and/or PGV application could be applied for sections of pipe through long vaults, on bridges or where inertial response might be important.

A pipe designed by the finite element method will often be shown on contract drawings showing material selection, joint preparation, trench design and other factors. An engineer's certified stress report may accompany an important pipeline designed by the finite element method.

C7.2 Chart Method

Due to the inherent assumptions in the Chart Method, the reliability / factor of safety / margin of the pipeline will not be quantified. Note that the ESM or FEM methods can be used at any time, and designs using the ESM or FEM methods will be more quantified than those based on the Chart Method. If there is a conflict between the Chart and the ESM or FEM methods, the method which provides the most confidence in meeting the overall performance goals should be relied upon. The ESM method will, in general, provide more confidence than the Chart method. The FEM method will, in general, provide more confidence than the ESM method.

Tables 7-1 through 7-19 provide a simple classification system for pipelines versus the level of seismic PGV and PGD hazards. Once the PGV and PGD is estimated for particular pipeline location, then the designer uses the following tables to indicate the desired style of pipeline design. Note: the ESM or FEM methods can be used at any time, and the selections using those methods will always supersede the selection based on the chart method.

Table 7-3 deals with PGDs along the length (parallel) to the pipeline. These have been shown to cause more damage to pipelines than PGDs transverse to the pipeline, given an equal amount of PGD. Type E design is the same as Type D design, except with peer review.

C7.2.1 Design Approach

Tables 7-11 through 7-19 describe what the Guidelines mean for each design category. The end user can adjust these design categories by verifying (by test, ESM or FEM) that show that the seismic performance for a particular style of pipeline installation will meet the overall system-wide intended performance goals.

Tables 7-11 and 7-12 suggest that ductile iron pipe can be used for classifications D and E, whereas PVC cannot. It should be pointed out that PVC likely has superior corrosion resistance than ductile iron pipe, and this might be a trade-off for pipe selection. With suitable design, PVC pipe could be made able to tolerate large PGDs, by a combination of suitable joint restraint devices, shortness of pipe barrel length, etc. Nothing in these Guidelines should be taken as a recommendation to install one kind of pipe over another, as long as a rational analysis can be provided that shows the installed pipe will meet the intended performance.

C7.2.2 Distribution Pipelines

The authors of these Guidelines had considerable debate as to whether Function II pipelines having PGV>30 inches/second should be classified for design as "A with extra valves" or "B". A water utility having a high percentage of pipelines located at sites subject to intense shaking (PGV > 30 inch/sec) at 475-year return period might wish to adopt superior pipe materials at such locations. The Guidelines suggest only that extra valves be inserted in such pipelines so as to minimize the number of customers having to be isolated should the pipe require repair.

Should the owner conduct a system-wide vulnerability study and determine that the overall damage level (from PGV and PGD mechanisms) results in unacceptable system performance and restoration times, then it might be prudent for the owner to increase the design requirement for distribution pipelines from A to B at the highest levels of ground shaking.

C7.2.4 Design Approach

The "standard with bypass" option is listed only for pipes that are likely to be exposed with substantial PGDs, and be Function Class III or IV. There is no good way to bypass damage to thousands of broken distribution pipeline that is cost effective. Installation of hoses for bypass purposes post-earthquake requires suitable valving and outlets, suitable lengths of hose of the right diameter, and significant manpower and equipment for deployment.

If only a few houses are out of water in an entire system, then use of 2-inch diameter hose to connect to hose bibs at individual houses has been done in past earthquakes. The authors of these Guidelines do not envision that this strategy will be workable for possibly many thousands of structures in a modern urban environment; instead, we suggest that the distribution pipes be suitable designed and installed so as to preclude widespread damage the first place.

C7.3 Equivalent Static Method

The ESM makes a number of simplifying assumptions, and it should be understood that the ESM cannot completely account for particularly unusual ground conditions or pipeline configurations. The ESM presented in the Guidelines reflects concepts presented in (O'Rourke and Liu 1999, O'Rourke, Wang and Shi, 2004, JWWA 1997, ASCE 1984) and other sources.

The ESM can be always augmented by refinements in defining of the hazard, the analytical technique and the design of the pipe. Given the simplifying assumptions, variability and uncertainty in the hazard description, soil conditions, analytical techniques, pipeline capacities, as well as the underlying goal that some system-wide damage is acceptable, refinement in the ESM may (or may not) not be warranted. For important pipelines (Function Class III and IV) and where the PGD hazard is well characterized (total displacement and deformation pattern), consideration should be given to use of the Finite Element Method.

As of 2005, quantified strengths and displacement capacities of pipes and pipe joints are not usually included in pipe manufacturer's catalogs. Pipeline designers need such information to make informed decisions as to pipe selection for particular installations. One approach that a designer can take is to put the required pipeline forces and displacement capacities into a specification, and allow the pipeline vendor to supply that information as part of the procurement process.

C7.3.1 Analysis for Ground Shaking Hazard

In practice the most energetic seismic waves in common soil conditions are shear (body) waves, and these can propagate at speeds of c= 12,000 to 20,000 feet per second. In uncommon cases, less energetic Rayleigh (surface) waves can propagate at slower speeds.

There is open question as to the actual energy of body and surface waves and their propagation speeds; however there is strong evidence that wave propagation (PGV) loading without concurrent PGD loading causes just limited or modest damage to buried water pipe networks; at least in past earthquakes in coastal California.

To simplify these Guidelines, we just assume c = 13,000 feet / second as a safe design approach in most instances. The user can always perform site-specific studies to refine this assumption for sites with special characteristics.

The pipe barrel should be designed to remain elastic (such as for steel and ductile iron pipe) for ground shaking. For materials where yield level is not applicable, the design for the pipe barrel should have very high reliability against failure under the 475-year ground shaking motion. For metal pipes, pipe barrel yielding due to ground shaking should be avoided unless the underlying system-wide performance goal is assured.

Continuous Pipe

For continuous pipe, the seismic ground strain is accommodated by alternating axial tension and axial compression in the pipe. If the wave length of the seismic excitation, λ, is long and the soil is strong (large ultimate force per unit length at the soil pipe interface, t_u) the axial strain in the pipe is about equal to the ground strain. Hence, the axial force in the pipe is the ground strain times the pipe axial rigidity and the peak force is computed as F_1. On the other hand, if λ is short and/or t_u is small, the axial strain in the pipe will be much less than the ground strain. The maximum force in the buried pipe is t_u times a quarter wavelength "development length". This is the peak force F_2.

For example, given that PGV = 50 cm/sec and c = 13,000 ft/sec. Then:

$$\varepsilon_{pipe} = \frac{50}{13{,}000x12x2.54} \quad \frac{cm/\sec}{(ft/\sec)*(in/ft)*(cm/in)} = 0.000126$$

For a continuous pipe, (like double lap welded steel pipe), then the peak seismic stress along a long straight length of pipe is (but not at the joint):

$$\sigma_{pipe} = \varepsilon_{pipe} E = 0.000126 * 29000 ksi = 3.7 \;\; ksi$$

This modest level of axial stress in the pipe due to ground shaking is much less than the nominal yield stress of steel (depending upon grade, 30 ksi or higher). This example demonstrates that even moderate to strong levels of ground shaking should not cause much, if any, damage to continuous welded steel water pipelines, even if they use just single lap welded joints.

Assume a 43 inch outside diameter steel pipe with wall thickness of 0.50 inches. The pipe axial area is about:

$$A = \pi D t = 3.14 * 42.5 * 0.50 = 66.7 \;\; sq. \; inches$$

Assuming the pipe does not slip through the soil, the peak pipe axial (tension or compression) force is then:

$$F_1 = 66.7 * 29000 * 0.000126 = 244 \;\; kips$$

For the example pipe buried with in medium stiff clay, and assuming a typical concrete coating system, then t_u (see Section 7.4 for details on calculating t_u) is about 938 pounds per inch.

$$F_2 = \frac{0.938 * 6{,}500 * 12}{4} = 18{,}300 \, kips$$

The recommended design force for this example is therefore 244 *kips*.

Continuous Pipeline with One Unrestrained Joint

This model in Figure 7-3 can be used for a long welded steel pipeline with a single dresser coupling (say near a valve), in order to size up the required expansion movement at the coupling. For example, say A = 170 square inches, E = 29,000 ksi, t_u = 0.89 kip/inch, V = 32 inches/second, c = 2000 ft/sec (assumes very soft soil conditions), λ = 2000 feet (assumes long period motions). Then T = 1 second, $\delta_o = 10.2$ inches, R = $4V/c\lambda = 2.22e-7$ and $t_u/EAR = 0.81$ and $\delta/\delta_0 \approx 0.5$; so $\delta \approx 0.5 * 10.2 = 5.1$ inches.

This simplified model ignores water thrust forces. All pipe should be designed to accommodate hydrostatic and hydrodynamic water thrust forces in addition to any forces or movements needed to accommodate strains from ground shaking or permanent ground deformations.

C7.3.2 Analysis for Landslide and Liquefaction Hazard

For landslides and liquefaction, the hazard is characterized as being either longitudinal (pipe axis more or less parallel to the direction of permanent ground movement), or transverse (pipe axis more or less perpendicular to the direction of permanent ground movement).

Buried Pipe Response to Longitudinal PGD

There are a number of different ground displacement patterns for longitudinal PGD. The relationship in these Guidelines is based upon a uniform block pattern (Figure C7-1). In a block pattern, a mass of soil having length L_s, moves a distance δ down-slope (or towards a free face). Procedures for establishing expected values for both the length of the soil block L_S, as well as the amount of ground movement δ are presented in Sections 4.3 and 4.4. In lieu of specific knowledge about the particular site, the values in Table C7-1 are suggested. The recommended value for Function Class II is taken as the median of the observed data for actual lateral spreads, while the values for Function Class III, and IV correspond approximately to the 70 and 90 percentiles respectively.

F_1 is based on the following assumptions for an elastic pipe.

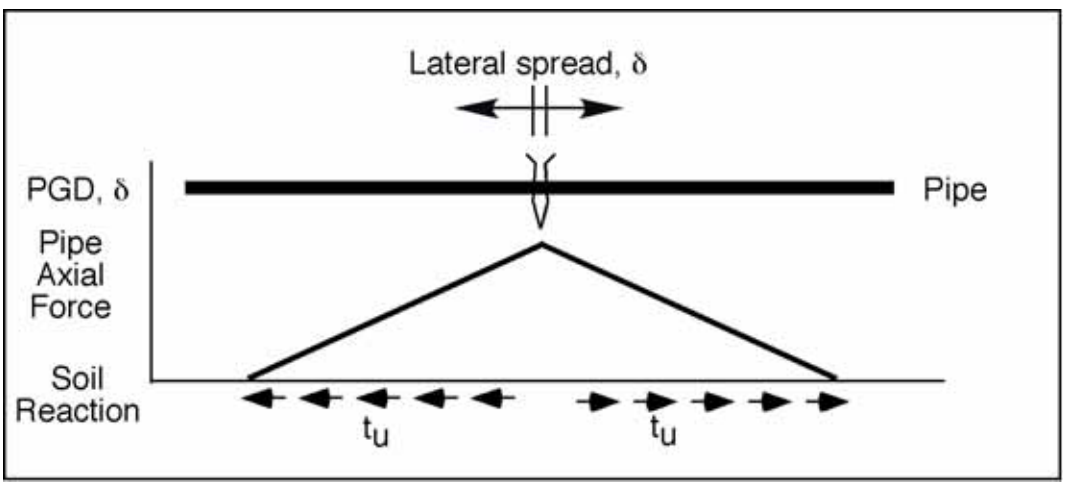


Figure C7-1. Idealized Lateral Spread

The total displacement to be absorbed by the pipe between the two end points is δ, or $\delta/2$ on each side of the spread, and the spread extends for a length L_s on each side of the ground crack in Figure C7-1. Then:

$$\frac{\delta}{2}=\int_0^L \varepsilon_p dx=\int_0^L \frac{t_u x}{AE}=\frac{t_u L^2}{2AE}$$

$$L=\sqrt{\frac{\delta AE}{t_u}}$$

$$F_1 = Lt_u = \sqrt{\delta AEt_u}$$

The user should recognize that the displacements in Table C7.1 are *scenario* displacements, meaning that they assume the site *will* liquefy and *will* have a lateral spread (or, the landslide will move). In practice only a percentage (often between 5% and 50%) of an area that is mapped as having high to very high liquefaction (or landslide) susceptibility actually will liquefy and move in a lateral spread, given a large earthquake with sufficiently high acceleration and sufficiently long duration so as to analytically predict that liquefaction might occur. Therefore, for design purposes for a complete distribution network, when relying upon incomplete subsurface information, the Chart Method (which already incorporates probability that the hazard occurs) might provide a first-order solution; or if using the ESM, some factor should be considered to consider the probability of lateral spread on an individual pipe and design accordingly. If the ESM approach is used, and if the subsurface information is largely unknown (except that the pipe is located in an area with high to very high liquefaction/landslide susceptibility), then a rational design might be to multiply the scenario-based spreads (listed in Table C7-1) by about 0.20 (or P_{ml} per Table C4-4) and then design using that displacement. For Function Class III and IV pipes, their importance would suggest that suitable geotechnical investigations be performed, and using the scenario-based design motions is appropriate.

Function Class	L_S (ft)	δ (ft)
II	300	6
III	500	9
IV	700	15

Table C7.1 Recommended Values for the Length of the Longitudinal PGD Zone, L_S, and the Amount of Ground Movement δ

<u>Continuous Pipe.</u> Longitudinal PGD results in areas of axial tension and axial compression in continuous buried pipe. If the length of the block L_S is relatively large, there are separate regions of axial tension near the head of the slide and axial compression near the toe. Between these regions, that is near the center of the block the axial stress in the pipe is zero and there pipe displacements match that of the ground. For elastic pipe it can be shown that the peak force (tension at the head and compression at the toe) needed to cause the pipe to stretch a displacement, δ is F_1.

If the length of the block is relatively small, the regions of axial tension and axial compression will abut each other, and the pipe displacement at the center of the block will be less than δ . For this case, the peak axial force in the pipe (tension at the head, compression at the toe) is due to the soil friction force is F_2.

When L_S is relatively small, the length of the block controls, and F_2 gives the peak force in the pipe, and F_1 overestimates the peak force. Conversely, when L_S is large, the amount of ground movement δ controls, F_1 gives the peak force in the pipe, while F_2 overestimates the peak force. Hence, it is appropriate to use the smaller value of F_1 or F_2 as the design force.

Segmented Pipe. Longitudinal PGD results in axial expansion and contraction at the joints of a segmented pipeline. For a block pattern, joints in the immediate vicinity of the head and toe must accommodate the PGD movement δ . For pipe systems with unrestrained joints, it is assumed that the ground movement δ is accommodated by expansion of a single joint at the head and by contraction of a single joint at the toe.

For pipe systems with restrained joints (chained joints = the joint can slip somewhat, and then a restrained stop restricts further movement), it is assumed that (n + 1) pipe segments and n restrained joints at both the head and toe of the longitudinal PGD zone accommodate the ground movement δ . Hence each restrained joint must allow δ / n worth of expansion at the head or δ / n worth of contraction at the toe. In order for a restrained joint to "share" and "distribute" the imposed ground movement, it must be able to transmit axial force in its fully expanded or fully compressed state. For n restrained joints near both the head and toe regions, the axial force in the joint increases as one moves closer to the head and toe, respectively. The axial force in the joint closest to the head and toe is F_{stop}.

A factor of safety of 2 is suggested for design of the stop in tension, recognizing that the stop might weaken over its lifetime (corrosion), there may be installation defects, etc; if the designed can demonstrate otherwise, the factor of safety can be reduced to 1.25. The stop need not be stronger than the actual yield of the barrel in tension.

In compression, the stop mechanism of a chained joint might be the male spigot bearing against the female end, such as for ductile iron pipe. For PVC pipe, there may be no "stop" in that the male spigot might be able to squeeze into the adjacent pipe barrel; in such a case, it would be good to confirm that adjacent barrel does not split.

Buried Pipe Response to Transverse PGD

Equations [7-14, 7-15, 7-16] in the Guidelines are based upon a sinusoidal pattern of ground displacement (y(x)) across the PGD zone, that is

$$y(x) = \frac{\delta}{2}\left(1 - \cos\frac{2\pi x}{W}\right)$$

where, W is the width of PGD zone and δ is the amount of transverse movement towards the center of the zone.

Procedures for establishing expected values for both the width W of the zone for transverse PGD, as well as the amount of ground zone movement δ are presented in Sections 4.3 and 4.4. The *scenario* values in Table C7-2 are suggested if the hazard is confirmed by suitable investigation; probabilistic values can be used if the hazard is only roughly defined in terms of its location and likelihood of movement at the 475-, 975- or 2,475-year return period motions. The recommended value for Function Class II is taken as the median of the observed data, while the values for Function Class III and IV correspond approximately to the 70 and 90 non-exceedance percentiles, respectively. Note that for transverse PGD, the hazard is more severe for pipes for smaller values of W.

The user should recognize that the displacements in Table C7.2 are *scenario* displacements, meaning that they assume the site *will* liquefy and *will* have a lateral spread (or, the landslide will move). In practice only a percentage (often between 5% and 50%) of an area that is mapped as having high to very high liquefaction (or landslide) susceptibility actually will liquefy and move in a lateral spread, given a large earthquake with sufficiently high acceleration and sufficiently long duration so as to analytically predict that liquefaction might occur. Therefore, for design purposes for a complete distribution network, when relying upon incomplete subsurface information, the Chart Method (which already incorporates probability that the hazard occurs) might provide a first-order solution; or if using the ESM, some factor should be considered to consider the probability of lateral spread on an individual pipe and design accordingly. If the ESM approach is used, and the subsurface information is largely unknown (except that the pipe is located in an area with high to very high liquefaction/landslide susceptibility), then a rational design might be to multiply the scenario-based lateral displacements (listed in Table C7-2) by about 0.20 and then design using that displacement. For Function Class III and IV pipes, their importance would suggest that suitable geotechnical investigations be performed, and using scenario-based design motions is appropriate.

Function Class	W (ft)	δ (ft)
II	900	6
III	700	9
IV	500	15

Table C7-2 Recommended Scenario Values for the Width of the Transverse PGD Zone, W, and the Amount of Ground Movement δ. (Lateral Displacement)

Transverse displacements due to liquefaction-induced settlement can be based on Table C4-5; these will be much less than those in Table C7-2 (spread). The approach in Section C4.6.1 can be used to establish the value δ in lieu of Tables C7-1 and C7-2 for purposes of lateral spread.

<u>Continuous Pipe.</u> A continuous buried pipe subject to distributed transverse PGD will tend to follow the soil displacement by bending in the horizontal plane; some pipe slippage will usually occur. For the sinusoidal pattern assumed, ground movement results in negative bending moments at the margins of the zone, and positive moment at the center of the zone. In terms of its flexural behavior, the pipe behaves like a fixed-fixed beam subject to a transverse load. In equation [7-14], it is assumed that the pipe follows the ground displacement exactly. In equation [7-15], it is assumed that the pipe acts like a beam carrying the load. Since both are limiting conditions, the prescribed strain is the smaller.

<u>Segmented Pipe.</u> For segmented pipeline systems with unrestrained joints, transverse PGD is accommodated primarily by a combination of axial expansion and angular rotation at the joints. The joint axial expansion arises from arc length effects. That is the total length along the deflected pipeline is larger than that for the originally straight pipe. The joint angular rotation results from the nominally rigid ($EI = \infty$) pipe segments mimicking the transverse ground displacement. The peak axial expansion due to arc length effects occurs at different points than the peak angular rotation. The maximum joint openings due to the combined axial and rotational effects are described by the equations in the Guidelines are adopted from O'Rourke and Nordberg (1991).

Segmented Buried Alternate Method

The ESM method to design pipelines to accommodate liquefaction-induced PGDs relies on assumptions about the general nature of PGDs in liquefaction zones. During the 1995 Hyogo-ken Nanbu (Kobe) earthquake in Japan, large amounts of liquefied ground moved towards the sea (downslope) when retaining walls at the ground/sea interface failed and rotated towards the sea. The movement was a lateral spread. The spreading caused significant damage to buried water pipelines.

The recommended approach is as follows.

First, determine the liquefaction susceptibility of the area where the pipe will traverse. Regional maps such as that shown in Figure C4-3 are a good source. Maps such as these are available on-line (http://www.abag.ca.gov/bayarea/eqmaps/liquefac/liquefac.html) and USGS in GIS format, and can be expanded to show particular city streets.

For areas mapped as having "high" or "very high" liquefaction susceptibility, assume that a percentage of such mapped areas will liquefy in earthquakes with M 6.5 of higher, when the fault is within 20 km of the site. The percentage of land that will liquefy will depend on local soil subsurface conditions, ground water table, etc. In the ESM method, developing such detail is not required. Instead, the following simplifying assumptions are made:

- At locations that do liquefy, and are located within 1,000 feet of a water boundary (bay front or creek) or on land with average slope more than 1%, the resulting ground strain, ε_g, in the downslope (toward the water) horizontal direction will typically range from 0.5% to 1.0% (60% of locations) and up to 2.0% (90% of locations). The suggested design value of 1.5% is a reasonable estimate of high (but not highest) ground strain.

- At locations that do liquefy, and are located more than 1,000 feet from of a water boundary (bay front or creek) or on land with average slope from 0% to 1%, the resulting ground strain, ε_g, in any horizontal direction will typically range from 0.5% to 1.0% (75% of locations) and up to 1.5% (90% of locations). The suggested design value of 0.75% is a reasonable estimate of high (but not highest) ground strain.

For example, at a relative flat location more than 1,000 feet from a shoreline, for a pipe with lay length of 12 feet, the pipe joint movement is predicted to be 0.0075 * 12 feet * 12 in/ft = 1.08 inches at the joint. This alternate method requires chained segmented pipe with designed stops or continuous pipe with suitable joints.

Just because the map in Figure 4-1 shows an area as having high or very liquefaction susceptibility does not mean that it will actually liquefy, even in large magnitude earthquakes. If the designer wishes to do careful subsurface investigation for the pipeline alignment, and finds soil layers susceptible to liquefaction, then more accurate and refined designs can be accomplished. However, this level of detail will not often be employed for small diameter distribution pipelines, and probably never for service laterals.

As a compromise between level of analysis / subsurface investigation and cost, we suggest the following approach for segmented pipe:

- Select Function Class
- Estimate the PGD. The PGD varies based on Function Class.
- $\Delta_{design} = \Delta_{joint} + \Delta_{operational} + 0.25 \ \ inch$

C7.3.3 Fault Crossing Ground Displacement Hazard

Fault crossing is arguably one of the most severe hazards for buried pipe. The horizontal and vertical offsets can be large (2 to 3 feet for magnitude 6 to 6.5 earthquakes, and 10 feet or much more for M 7.5 and larger earthquakes), and occur over relatively narrow fault zone. The relations presented in the guidelines are based on the conservative assumption that the offset occurs across a single line (i.e. “knife edge” fault). Hence, the hazard is simply characterized by the offset δ. Procedures for establishing appropriate values for δ are presented in Section 4.5.

Continuous Pipe

The Newmark - Hall (1975) closed form method to estimate pipe strain due to fault offset has been shown by finite element, empirical and test methods to ignore an important failure mechanism, that is, the localized bending and possible wrinkling in a continuous pipeline within 20 to 50 feet either side of a fault offset. As the formula, equation [7-18] without the first "2", is very easy to use, only requiring estimates of t_u, L_a and the amount of fault offset δ, it is retained in these Guidelines, but increased by a factor of 2. This is not to say that we endorse the method or its findings for other than a quick estimate of pipe strain for a given amount of fault offset. For important pipelines, this method should only be used for initial sizing purposes; and the FEM method should be use to design validate the pipe. Further, it is recommended that this formulation only be used if the pipe is subject to net tension, as the formulation ignores "p-delta" type effects when the pipe is subject to net compression.

A steel pipe with double lap welds can be used to accommodate fault offset. The double lap welds invoke a stress and strain riser, such that the girth joint will begin to wrinkle at about 90% of nominal yield in the main pipe (if in compression) or accumulate peak strain much faster than the main body of the pipe (if in tension). In tension, a common double lap welded joint might fail one-third of the time when the main body of the pipe has reached 8% strain (due to welding flaws and geometric intensification).

Segmented Pipe

For segmented pipeline systems with unrestrained joints, the fault offset is accommodated by axial expansion/contraction and angular rotation at the joints in combination with bending at the pipe segments between the joints. The relation presented in the Guidelines assume that the two joints closest to the line of rupture (one on each side of the fault) accommodate all the offset. That is, it is assumed that the joints are incapable of transmitting axial tension, axial compression or bending moments.

The fault offset δ can be decomposed into a longitudinal component $\delta \cos\beta$, parallel to the pipeline axis and a transverse component $\delta \sin\beta$, normal to the pipeline axis. The relations in the Guidelines for the required axial extension/contraction capability are based upon the assumption that the longitudinal component is shared equally by the pair of joints, each side of the fault line. The Guideline relations also assume that the transverse component of fault offset is accommodated by angular rotation of the same pair of joints. The pipe segment that crosses the fault rupture line is subject to shearing forces from soil pushing in one direction on one side of the fault, and pushing in the opposite direction on the other. The relations in the Guidelines for moment and shear are based on the assumption that the center of the pipe segment is located directly over the fault.

As a matter of practicality, segmented (unchained) pipe will likely fail when subject to fault offset much over a few inches to at most a couple of feet. Suggested design is a continuous pipeline with joints capable of sustaining considerable yielding; pipe bodies that are not subject to much (if any) wrinkling; or, possibly in lesser important pipelines, chained joints.

C7.4.1 Pipe Modeling Guidelines

The effects of internal pressure (up to about 150 psi, typical for water pipelines) on the behavior of pipes to withstand PGDs such as fault offset has generally been shown to have the following impacts:

- Internal pressure will tend to lower the axial forces needed to initiate wrinkling
- Internal pressure will tend to increase the capability of the pipe to withstand extended wrinkling once it has occurred.
- Internal pressure will have negligible effect on total pipeline response when hoop stress caused by internal pressure is less than about 25% of the yield stress.

For regular steels (such as SA106 Grade B, A-53, A36, X42), the pipe material law might be described in a three-way piece-wise linear manner (a tri-linear stress-strain behavior): linear stress-strain relation up to nominal yield; then a reduced tangent modulus up to nominal allowable strain; and then a further reduced tangent modulus up to ultimate uniform strain.

Practitioners in the oil and gas industry suggest that pipe elements need not be shorter than one pipe diameter near locations of high bending in the pipe. For very large diameter water pipes (like 8 feet diameter), this discretization may be too large to capture the rapid changes in curvature near fault offset locations.

C7.4.2 Soil Modeling Guidelines

In most cases, soils can be modeled as bilinear load-deflection curves to capture the pipe-sol response.

Soil spring properties (stiffness, strength) should be varied to considered the likely range of field conditions, in order to get the upper bound /lower bound loads on the pipe and nearby appurtenances. Stiffer and stronger soils will usually result in higher pipe response (higher strains) at the PGD offset; but lower loading on the pipe away from the PGD offset; the opposite occurs for less stiff and weaker soils.

The soil strength descriptions in Figures 7-6 through 7-11 are also included in ALA (2001), presented by formulae instead of charts.

C7.4.3 Wrinkling

The "wrinkling strain" is usually reported in the literature as the strain in the main barrel of the pipe at a distance away from the wrinkle. In fact, once the pipe starts to wrinkle, the actual strains in the wrinkle will be much higher than those in the main barrel of the pipe away from the joint. Equations [7-31 and 7-32] provide allowable strains in the main barrel of the pipe away from the wrinkle. Equation [7-31] (without the 0.75 reduction factor) assumes the D/t ratio is less than 120 (Gresnigt, 1986) and is based on the strain at maximum moment capacity at the wrinkling. If a complete nonlinear analysis of the pipe is done, then a suitable spring/finite element formulation of the wrinkle should show unloading in the main barrel of the pipe away from the wrinkle, while strain builds up rapidly within the wrinkle, as PGD is increased. Simpler beam-on-inelastic-foundation type models will not capture this effect. In cases where the wrinkle is actually modeled, the strain allowable within the wrinkle is higher than those inferred by equations 7-31 and 7-32. Depending on application, the allowable strain within the wrinkle could be as low as 5% (high confidence that the pipe will not leak) to as high as 20% (likely that the pipe will split open). It is left to the user to define a suitable strain within the wrinkle that matches the target performance for the pipe, should the acceptance criteria be based on strain within the wrinkle.

The Thames Water Pipeline (2.2m diameter butt welded steel pipe) underwent 3 m of right lateral offset in the August 17, 1999 earthquake on the Anatolian fault. Post-earthquake analyses of the pipeline (Eidinger 2001, Eidinger, O'Rourke, Bachhuber 2002). The pipe crossed the fault such that substantial compression and bending occurred in the pipe, and the pipe wrinkled. Figure C7-2 shows one of the wrinkles, as seen from inside the pipe. While the pipe leaked at one of the wrinkles, it remained in service for several days after the earthquake.

As measured inside the pipe, the wrinkles were from 5 inches deep to more than 20 inches deep. One of the results of these wrinkles was that there was an additional friction loss in the pipeline. Ultimately, due to reduced hydraulic capacity of the pipeline, the wrinkled section of the pipe was removed and replaced with two smaller diameter pipes in order to maintain overall hydraulic capacity of the pipeline.

Figure C7-2. Wrinkle of 2.2 Meter Diameter Thames Pipeline

C7.4.4 Tensile Strain Limit

The ultimate uniform tensile strain limit for thin walled mild steel (such as t=0.25 inches) is usually in the range of 20% to 22% or so. The ultimate uniform tensile strain is *not* the same as the strain at rupture, which might often be 30% or more. For thick walled steel (such as t=1 inches), test data might show lower ultimate uniform strain capacity. The recommendation to limit tensile strains to 0.25 times the ultimate uniform strain capacity is intended to provide for normal variations and provide some margin. If thought to be important, the designer can require that suitable plate tension tests be performed for the steel used for the pipe, and then set the allowable tensile strain limit at a suitable level below the actual test failure level. The factor of safety to be used should consider the desired reliability of the pipe, variation in test data, etc.; but should always be at least 2 (i.e., allowable tensile strain = 0.5 times ultimate uniform strain) if the designer wishes to retain at least some reliability for uncertainties and randomness that are not otherwise incorporated into the total design process.

The tensile strain limit should also be set in consideration of the weld procedures used. It is recommended that field-made girth welds shop welds in the pipe should have weld material strength (yield and ultimate) that exceeds the pipe strength (actual strength, not specified minimum), wherever nonlinear response of the pipe is expected. These Guidelines do not provide detailed welding design and installation procedures.

Honegger and Nyman (2004) propose that the tensile strain be limited to 2% to 4% for oil and gas pipes. These limits reflect concern over fracture toughness of steel. For water pipelines kept at reasonably high temperatures (typically 50°F or higher), brittle fracture is not the common failure mode, and a small leak in a water pipeline under a rare earthquake will usually be acceptable. Thus, for water pipes, the allowable tensile strain can usually be set in the 4% to 5% range. In any case, a good design for a water pipeline that crosses a fault is to keep the tensile strain in the pipe at around 2% or so, given the offset and median soil properties.

C8.0 Transmission Pipelines

Analytical formulations such as those presented in Section 7 would suggest that for an equal amount of imposed ground strain, a large diameter pipe should experience the same strain as a small diameter pipe. If repair rate is only a variable of ground strain (as has suggested using simplified fragility models), then there should be no observed difference in repair rate between small and large diameter pipes.

Since it has been observed in real earthquakes that large diameter pipelines usually perform better than small diameter pipelines, it might be concluded that imposed ground strain is not the only parameter of importance. Other factors, such as corrosion, quality of construction, presence of laterals, hydrodynamic loading, etc. might all contribute to the actual failure mechanisms.

C8.1.2 Pipe Materials and Thickness

D/t ratios for welded steel pipe for water pipes are typically in the range of 150 to 225 for pipes sized only for internal working pressure. At fault crossing (or other PGD) zones, high D/t ratios are to be avoided, in order to provide for better nonlinear performance of the pipe. A maximum D/t ratio of about 90 to 100 is suggested, in order to provide for some compressive yielding prior to major wrinkling. At fault crossing locations, D/t ratios of about 50 have been used for smaller diameter (24-inch or so) butt welded oil and gas pipelines. For larger diameter pipes, the need for D/t ratios of 50 or so is possibly not cost effective, so the designed should strive to keep compressive forces (strains) in the pipeline as low as practical; a D/t ratio of 90 to 100 can provide a high capacity to take fault offset (or other sharply-applied PGD) with suitable care taken in the overall design process.

C8.1.3 Design Earthquakes

For high seismic hazard areas, the owner may wish to consider two levels of earthquakes that should be evaluated, if the owner wishes to have two levels of performance goals, such as:

- Extremely reliable under Probable Earthquake
- Reasonably reliable under Maximum Earthquake

The Maximum Earthquake represents an upper level that is unlikely to be exceeded during the remaining life of the pipelines; for Function IV pipelines, these Guidelines suggest the use of a 2,475 year return period probabilistic earthquake. In coastal California, the ground motion for a 2,475 year earthquake is very roughly about 50% larger than that for a 475 year earthquake.

The lower level, Probable Earthquake, represents an event more likely to actually occur during the pipeline's life. Response spectra and time-histories in displacement, velocity and acceleration need to be developed.

The Guidelines avoid the use of "importance factors" that are common to many regular building codes. Instead, the Guidelines retain the return period as the measure of acceptable risk tolerance for varying types of pipes by their importance to the pipe network, and then retain a constant design process for every kind of pipe.

Should the owner wish to use a two level design strategy, then it is up to the owner to establish the meaning of "probable" earthquake. For major transmission pipes (Function Class IV), the probable earthquake could be set at a return period of 100 to 475 years. For example, say a "fault memory" model is used, such that a major transmission pipe crosses an active fault with about a 1% chance per year of fault offset of a few feet. With such a high likelihood of fault offset of occurring in the planning horizon, the owner may wish assurance that the pipe will reasonably accommodate the median fault offset in such an event; as well as having a good reliability of accommodating an 84^{th}-percentile not-to-exceed offset that is contemplated using the simple multipliers in Table 4-6. The design of the pipe would follow these Guidelines, except that the allowable post-yield strains due to PGD would be half the values listed in the Guidelines.

C8.1.6.1 Welded Steel Pipe

Figure 8-1 shows one way to prepare a full penetration welded girth joint for a steel water pipe. This joint might be susceptible to damage unless care is given to the quality of the root pass. There are alternate methods to construct such a joint in the field. Whichever way is adopted, the joint should have suitable inspection and testing. Industry manuals of practice from API, AWWA, ASME and others address these issues.

C8.1.6.4 Reinforce Concrete Cylinder Pipe (RCCP) and Prestressed Concrete Cylinder Pipe (PCCP)

The joint type shown in Figure 8-11 (or similar versions) is commonly used for PCCP and RCCP pipe. The designer can specify that the joint should be welded closed after the pipe is installed, but before the cement mortar is placed in the field. A fillet weld is commonly placed between the two thickened bell rings. This fillet weld can take some tension force, but not enough to force general tension yielding of the pipe itself.

When there is a bend in a pipe, there will be a hydrostatic thrust on the bend. This thrust must be resolved by using concrete anchors on the bend, or by direct skin friction between the pipe and surrounding soil (tu). For very large diameter pipes, concrete anchor blocks are not often used. Instead, the common approach is to weld the joints closed.

The number of joints to be welded closed should consider the hydrostatic and hydrodynamic thrust loads on the bend. The hydrodynamic portion of the load can be estimated using the procedure outlined below, or by other rational methods. We do not recommend relying on the tensile capacity of the cement grout to resist any of these thrust loads. A sufficient number of joints should be welded to ensure that the hydrostatic thrust is can be resolved using t_u with a about a factor of safety of 3; or hydrostatic plus hydrodynamic with ideally a factor of safety greater than 1.25. As t_u is variable, and some minor joint cracking does not mean leakage, it is not obvious that a much higher factor of safety is warranted.

Sudden valve closures, pump trips and seismic wave passage will result in hydrodynamic loading in pipelines. In the past, hydrodynamic loading due to seismic loading has usually been ignored. For pipelines with welded joints, the effect of seismic-induced hydrodynamic loads is usually minimal, in that the hydrostatic design of the pipe will usually have sufficient factor of safety to withstand the short duration dynamic loads (but this should be checked).

Instances where hydrodynamic loads may be especially important include bends in transmission pipelines designed for low internal pressure (under 100 psi static), coupled with high ground shaking. The hydrodynamic load is a function of the mass of the water being excited along the length of the pipe, coupled with the propagation of the water pulse at the sonic velocity of water in the pipe. For steel pipelines, the velocity will usually be on the order of about 3,000 feet per second; for thick-walled concrete pipe, the velocity may be a bit higher (see Section 6.6 for computation of the wave velocity).

To establish a simple estimate of hydrodynamic loading, a finite element analysis was conducted of a 66-inch diameter steel pipeline that is straight for 20,000 feet, with a ninety degree bend at one end. The water in the pipeline is modeled using mass elements, with the "stiffness" of the water being adjusted to obtain a sonic velocity of 2,900 feet per second. A series of 18 different earthquake time histories was applied to the model. The peak hydrodynamic force at the bend was found to be best correlated with the spectral acceleration at T=2.4 seconds (5% damping) of the input motion (Figure C8-1). For peak water hammer pressure, the best fit curve suggests:

$$p_h \cong 0.85\left(SA_{T=2.4\,\mathrm{sec},\ 5\%\ \ damping}\right) \qquad \text{[Eq. C8-1]}$$

where p_h is the peak hydrodynamic pressure in ksi at the bend and SA is the 5%-damped spectra acceleration of the input motion at a period of 2.4 seconds, in g. For design, a reasonable approach will be to require restrained joints for a distance from each bend such that the combined hydrostatic + hydrodynamic thrust loads can be resisted by skin friction reactions (t_u) between the pipe and the surrounding soil. Along the length of the pipe, the peak hydrodynamic pressure will typically be about 50% to 80% of that at the bend. For practical situations where the hydrostatic pressure is 100 psi, and the design motion has PGA much less than 0.3g, the pipe should have adequate margin with withstand the seismic hydrodynamic loads. For situations where the pipeline has low hydrostatic pressure (say 50 psi), and is exposed to large earthquakes with long period motion, the hydrodynamic pressures can reach 300 psi or so, resulting in large thrusts at bends and pull-apart of unrestrained joints near the bend.

The designer is cautioned that the model shown as a straight line using the triangle data points in Figure C8-1 will vary based on pipe diameter and length between bends.

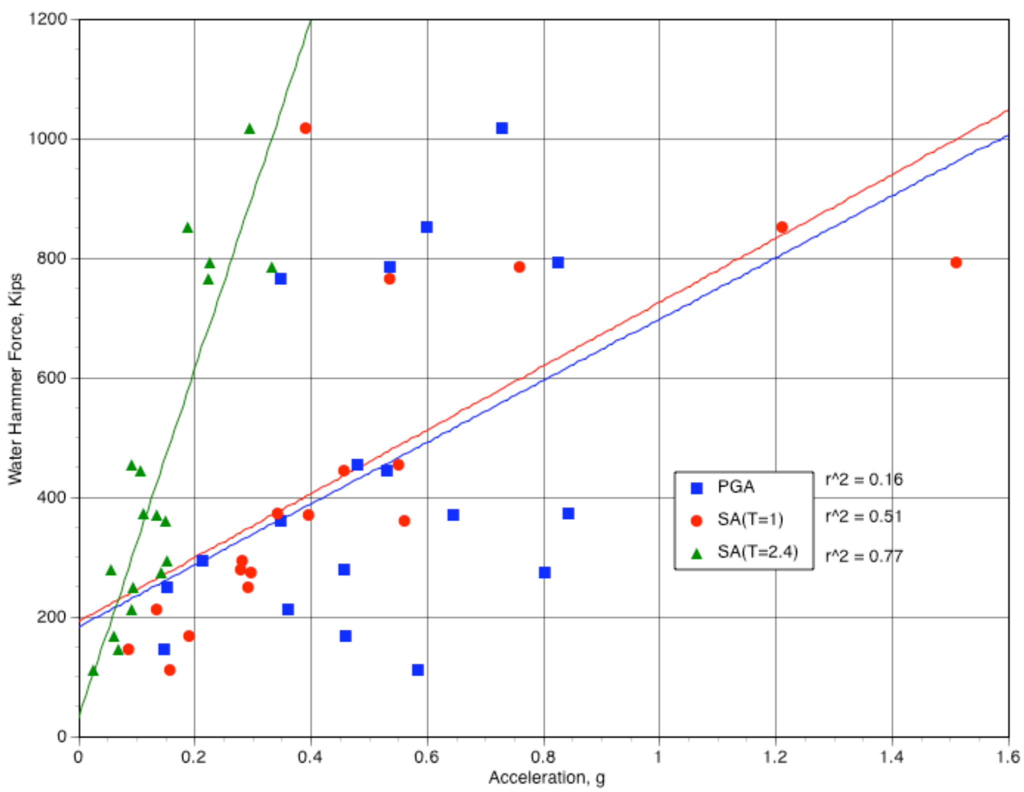


Figure C8-1. Hydrodynamic Water Hammer Force at 90-Degree Bend in Pipeline

PCCP has had a variable track record under seismic loading, with high repair rates in the 1994 Northridge earthquake (higher than cast iron, on a per-mile basis), but with much lower repair rate in the 1989 Loma Prieta earthquake (better than welded steel pipe; albeit with somewhat lower intensity of ground shaking, and mostly acting on relatively young (under 20 year old) pipe). One reason for the variable repair rate for PCCP between the 1989 Loma Prieta and 1994 Northridge earthquake could be that the pipes near San Jose (1989 earthquake) may have had more tension joints (a tension joint is Figure 8-11 with a fillet weld closure) than the pipes near Santa Clarita (1994 earthquake), and so direct comparisons may not be applicable. A dearth of tension joints near bends, low pressure pipe and strong long period pulses in the Northridge earthquake could be an important factor in explaining the differing performance.

C8.1.11 Isolation Valves

We recommend placement of isolation valves (usually gate or butterfly valves, usually manually operated) between high vulnerability and low vulnerability pipelines. For example, there should be an isolation valve on the lower-class pipeline at each interface between different-class transmission pipelines.

Isolation valves are relatively expensive for transmission pipelines. As transmission pipelines have few branch connections, isolation valves should be placed on the smaller diameter (and often lower class) branch pipeline. In-line isolation valves should be placed on transmission pipes at intervals to allow for suitable maintenance and inspection cycles; and adjacent to particularly high hazard zones should bypass systems be contemplated.

In zones with very high ground shaking (PGV over 30 inches per second), we recommend that isolation valves be placed at close intervals for distribution pipes (including four isolation valves at every cross, three isolation valves at every tee), such that smaller sections of the pipe network will be isolated should there be pipeline damage.

C8.1.14 Corrosion

When designing a pipeline, the issue of corrosion must be addressed. If not protected, or if improperly protected, the pipeline may eventually fail without earthquake, or fail at many places due to earthquake. To prevent this, a corrosion engineer should be consulted and proper corrosion protection should be implemented. Since protecting a pipeline against corrosion can only maintain the pipeline's current condition and not reverse the effects of corrosion, it is ideal to have a corrosion protection system in place when the pipeline is first buried so that the pipeline's initial condition is maintained. Assuming a good design, proper maintenance of this system is all that is needed to ensure the pipe does not fail (or at most only very rarely) due to corrosion.

C8.1.20 Emergency Response Planning

The recommended strategy to repair pipe starts with making repairs to source water facilities, and following to the smallest pipe. This approach recognizes that one cannot make repairs to downstream distribution pipe until upstream pipes are repaired and water is available to provide pressure to find damaged locations.

In practice, a water utility might try to repair pipe first in areas of highest economic value, such as central business districts, etc. Limited experience (Kobe, 1995) suggests that this strategy might ultimately result in a slower overall repair time, especially is there has been substantial upstream damage that is left unrepaired.

In general, a water utility will not know the extent of pipeline damage after an earthquake. System models using fragility formulations and hazard estimates could be helpful in forecasting in real time the extent of the damage, but even so, there will be considerable uncertainty in the actual amount and spatial location of the pipeline damage. The recommended repair strategy recognizes that one must have sufficient water volume and pressure to find downstream pipe leaks, and that repairing one pipe will often result in finding additional downstream leaks once the repaired pipe is re-pressurized.

Point (10) describes a pipe replacement program. At the heart of the problem will be exposure of non-seismic pipe exposed to sufficient PGDs (or very high PGVs) to result in a large number of pipe repairs that cannot be rapidly repaired. In using these Guidelines, we intend that new pipe be installed using suitable seismic design practices. These Guidelines do not require that older pipe, such as cast iron with lead-caulked joints, be replaced solely because better pipe materials are now available. If the designer performs a suitable cost-benefit study, a rational pipe replacement program of vulnerable pipelines can be established; we would expect the pipe replacement cycle would vary between utilities, owing to different local hazard conditions, different pipe repair capabilities and different community needs.

One large water utility, EBMUD, has about 4,000 miles of installed pipeline (as of 2005). About 1,000 miles of these pipelines are cast iron pipe, another 1,000 miles of these pipelines are asbestos cement pipe. EBMUD currently replaces about 8 miles of existing pipeline per year, suggesting a (roughly) 500-year pipe replacement cycle. On the surface, a 500-year pipe replacement cycle would appear much too long. However, the cost of pipeline replacement is very high, and the benefits accrued from reduced future earthquake damage must be balanced against the high initial capital cost.

A possible practical pipeline replacement strategy might factor in the following issues:

- Replace pipelines due to operational needs (increased demands, etc.) as needed.
- Replace pipe segments that have leaked (for any reason) more than 1 time in the prior 10 years.
- Replace pipelines that cannot sustain PGDs (all segmented pipe with push-on joints) that traverse areas with high to very high susceptibility to liquefaction and landslide, for any water utility exposed to PGAs over 0.20g once every 475-years.

For portions of the United States that have 475-year return period PGA level of 0.6g or less (and this covers essentially all of the USA), these Guidelines would not suggest that wholesale replacement of all cast iron, (or any other type of push-on jointed pipes) be replaced for seismic purposes as the sole reason for pipeline replacement.

C8.2.3 Design Earthquakes and Associated Magnitude of Fault Displacements

Throughout these Guidelines, we recommend design of pipes for one level of earthquake, either the 475-year, 975-year or 2,475-year motion, depending on the Function Class of the pipe. In many cases, the design may assume a particular characteristic magnitude of earthquake (deterministic), and then design a Function Class IV pipe to withstand the 84th-percentile non-exceedance offset at the strain limits described in these Guidelines; such a design should meet the intent surviving any fault offset that might be expected in about a 2,475-year interval.

Steel pipes with high D/t ratios (on the order of D/t = 200) will likely have excessive ovalization when subject to fault offset, even when buried in soft soil-type trenches. Even if such a pipe is designed to have tension only (no wrinkling) and otherwise has acceptable tensile longitudinal strains, high ovalization may occur, with possible wall buckling / snap through. For this reason, we recommend that D/t ratios be kept to no more than about 90 to 100 in the immediately vicinity of the fault offset, unless the design explicitly accommodates pipe ovalization. Depending on actual design parameters, the pipe wall can usually be thinned to about D/t=200 (or as needed for internal pressure) at a distance of about 80 pipe diameters from the fault offset location; that actual distance will depend on pipe material properties, trench design, and possibly other site-specific factors.

C8.2.6 Joints Used to Accommodate Fault Displacements

The use of mechanical joints to accommodate fault offset is a discouraged practice for oil-and-gas pipelines. There may be good reason for such discouragement. For example, the joint shown in Figure 8-17 has been in service to accommodate ongoing fault creep for under 15 years; yet one of the exterior rotation joints has already rotated sufficiently with concurrent pipe ovalization such that the exterior harness is relied upon to transfer further movement to the middle compression joint. This raises questions about the capacity of the rubber gaskets to maintain leak-tightness.

C8.2.7 Analysis Methods

Simplified methods, such as the Newmark-Hall (1975) procedure, do not capture the failure modes (wrinkling due to high local bending) for pipelines that cross faults. Such simplified methods should be used with care, it at all.

These Guidelines are not intended to cover all the parameters needed to design above ground pipelines. However, the basic principles in the Guidelines are adaptable for above ground pipes, by suitable incorporating the inertial and damping terms. The user is cautioned that the typical UBC-assumption of 5% damping for buildings is generally not applicable for welded steel pipes; test data for welded steel pipe usually shows actual damping of perhaps 2% to 4% when there is no yielding in the pipe-support system.

When buried pipe transitions to above ground pipe (such as for bridge crossings, or when entering a vault), care should be taken to ensure that the inertial response of the above ground pipe is suitably considered in the overall design process.

C10.0 Distribution Pipelines

C10.2 Ductile Iron Pipe

Empirical evidence of the performance of push-on joint ductile iron distribution pipe in (ALA, 2001) suggests that the repair rate for such pipe due to wave propagation is:

$$RR = 0.5 * 0.00187 * PGV$$

where RR = repair rate per 1,000 feet of pipe and PGV in inches/sec. The 0.5 factor in this fragility model reflects ductile iron pipe with push-on joints. The empirical evidence suggests that about 5 of 6 repairs due to ground shaking will be leaks, and 1 of 6 repairs will be full breaks. Thus, RR = 0.1666 * 0.5 * 0.00187 * 30 = 0.004673/1,000 feet. This is well within the target break rate for 6-inch diameter pipe of between 0.03 to 0.06 per 1,000 feet, and would be so even if all the repairs were breaks. As a PGV of 30 inches/second is a very intense level of ground shaking, this suggests that push on joints for DI (or PVC) distribution pipe will be adequate for essentially every water system. One would thus expect one break and five leaks per 214,000 feet of such pipe; assuming average pipe length of 16 feet, this corresponds to one break and five leaks in about 14,000 pipe segments.

For PGD-type loads, assuming even PGD = 1 inch, the repair rate using fragility models is much higher:

$$RR = 0.5 * 1.06 * PGD^{0.319}$$

where PGD is in inches. The 0.5 factor in this fragility model reflects ductile iron pipe with push-on joints. The empirical evidence suggests that about half the repairs due to permanent ground deformation will be leaks, and half will be full breaks. Thus, RR = 0.5 * 0.5 * $1^{0.391}$ = 0.25/1,000 feet, or 4 to 8 times higher than the target break rate for 6-inch diameter distribution pipe.

The authors of these Guidelines observe that the above fragility models (ALA, 2001) are based on empirical evidence tempered by engineering judgment. As more empirical evidence is gathered in future earthquakes, there is no doubt that these fragility models will be updated. For example, use of better concrete anchors (or local restrained near bends) should substantially reduce the repair rate for segmented low pressure pipes when subjected to ground shaking with high long period energy. Similarly, fragility models that relate to directly to ground strain rather PGV and PGD have merit, although at the current time, there is no simple way to analytically predict ground strain, as this requires a-priori knowledge of wave propagation speeds, wave lengths, ground crack patterns, etc.

C11.0 Service Laterals

The installation of customer service laterals remains one area of design that has received scant attention in the literature. Yet, a typical water utility serving 1,000,000 people will have more than 400,000 service connections in its system. Damage to service connections in the 1906 San Francisco earthquake arguably had as much impact to the ensuing fire conflagration as breakage of some of the larger distribution pipelines. Similarly, more water was lost via service lines in the 1991 Oakland Hills fire than was used to actually fight the fire.

The non-seismic aspects of service line connections are that they must be made in the field rapidly, often while the distribution pipeline is under pressure, and must be reliable for many years. There have been many styles of such installations, ranging from copper to various types of plastic. Experience of utilities has shown that some installations are simpler to install, have less potential for corrosion / stray current issues. However, with the possible exception of these Guidelines, there has been little industry-wide guidance as to seismic performance.

C11.4 Design For Transient Seismic Ground Strains (PGV)

In Table 11-3, we make the assumption that service laterals are relatively short (often 10 to 30 feet in length) up to the customer meter box. Also, for cases where the seismic hazard is low to moderate (PGV under 10 inch/sec), the induced strain into the lateral is particularly small, and thus even a corroded lateral will suffer an acceptably small repair rate. Once PGVs get to be appreciably high, we make the assumption that it is desirable to have available the entire cross section of the lateral (ie., no corrosion), and thus we recommend that the lateral be suitably protected.

C11.5 Design For Permanent Ground Displacement

We make the assumption that the service boot type installation shown can take perhaps a few inches of relative displacement between the main and the service lateral. We list 12 inches as a transition point in Table 11-4 to recognize that some of the PGD might also be taken up by the main.

C11.5.2 Fire Hydrant Laterals

If two Dresser-type coupling are placed about 12 feet apart, and each coupling can rotate about 2 degrees without failure, then the total offset available (and assuming no damage to the pipe barrel) is about: 0.034 radians * 12 feet = 5 inches if the PGD is concentrated between the two couplings; or somewhat more if the PGD is concentrated beyond the two couplings. If the sense of PGD is axial along the lateral (like a hydrant placed in a slide on the fill side of a road, while the pipe is in the stable cut side of the road), then the couplings should be restrained.

C12.0 Other Components

C12.2 Equipment Criteria

The formulation of F_p is based on elastic response of equipment. We strongly advise against using response modifiers / ductility "knock-down" factors commonly used in codes such as the 1997 UBC, 2000 IBC or 2003 IBC (or, use them with $R_p = 1.0$). We doubt there are many cases when it is cost-effective to reducing the real forces to account for nonlinear performance of equipment. Nonlinear performance implies increased displacements and distortions, both of which can have negative impact of equipment operability. Since the bulk of the cost to properly anchor equipment is usually the installation labor, there is often no material cost-penalty to require anchor bolts and restraint hardware that is a sufficiently strong. The factor C_f could be as high as 2.5, but we adopt 2.0 reflecting that this would capture the median response including higher modes, for most installations. The factor C_g reflects that the PGA from the USGS web site is for the free-field surface, and there is usually considerable reduction in motion for floors in buried vaults.

C13.0 References

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