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July 17, 1981
EF2-54096

Mr. L. L. Kintner
Division of Project Management
Office of Nuclear Reactor Regulation
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Dear Mr. Kintner:

Reference: Enrico Fermi Atomic Power Plant Unit 2
NRC Docket No. 50-341
Subject: Buried Pipe Analysis
Shear Wave Velocity

In our letter of June 23, 1981 EF2-53866, we stated that at Fermi 2, an apparent shear wave velocity of 2500 ft/sec. has been used .

The 'state of the art' design methodology in this area is based on the work of Drs. Newmark and Hall (1) carried over some period of years in connection with design studies for the Trans-Alaska pipeline, the Canadian Arctic gasline, the Sohio pipeline from Long Beach, California to Midland, Texas, as well as other special facilities. The shear wave velocities to be used in design as suggested in ref. (1) are: 4000 ft/sec. for rock or permafrost, 3500 ft/sec. for massive gravel deposits, 3000 ft/sec. for sand and competent soils, and slightly lesser values for silt and clay deposits.

At Fermi 2, the buried pipes were installed in compacted rockfill. As per ref. (1) a shear wave velocity of 3000+ ft/sec. could have been applied. For the backfill compaction densities as placed in the field, the use of design shear wave velocity of 2500 ft/sec. at Fermi 2 is compatible with the state of the art.

Sincerely,

William F. Colbert
Technical Director
Enrico Fermi 2

WFC:dah

Attachment

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Seismic Design Criteria for Pipelines and Facilities*

By William J. Hall,[1] F. ASCE and Nathan M. Newmark,[2] Hon. M. ASCE

(Reviewed by the Technical Council on Lifeline Earthquake Engineering)

Seismic Design Philosophy

The design criteria and recommendations described herein take into account the seismic motions and seismic generated forces that have a reasonable degree of probability of occurrence along the route of a pipeline. The basis for the selection of these criteria and recommendations involves consideration of the acceptable risk of exceeding the design levels for the pipeline system and various classes of associated structures, equipment and facilities. For the most critical classes, where failure, defined as exceeding the allowable recommended levels, would have a bearing on life and safety of the population or might adversely affect the environment, or where for economic reasons interruption of the service provided by the pipeline is not tolerable, the margins of safety implicit in these criteria are often greater than those now used in the seismic design of major buildings in highly seismic regions of the United States. For the least critical classes, the margins of safety are at least as great as those provided by current building codes such as the Uniform Building Code or the Structural Engineers Association of California (SEAOC) Code (15). The procedures outlined will result in a design having appropriate factors of safety against seismic disturbances when combined with the other applicable operating and environmental conditions.

In accordance with principles developed for use in the design of nuclear reactor power plants, the design criteria generally encompass two levels of earthquake hazard. The lower level is that associated with a return period for the design earthquake of approx 50 yr–100 yr and is designated herein as the

Note.—Discussion open until April 1, 1979. To extend the closing date one month, a written request must be filed with the Editor of Technical Publications, ASCE. This paper is part of the copyrighted Journal of the Technical Councils of ASCE, Proceedings of the American Society of Civil Engineers, Vol. 104, No. TC1, November, 1978. Manuscript was submitted for review for possible publication on March 9, 1978.

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"Design Probable Earthquake." The higher level is that associated with a longer return period, of the order of about 200 yr–300 yr or more, and is designated as the "Design Maximum Earthquake." Under some situations it may be expedient to use only one such level, generally the latter.

Conceptually one might consider the first earthquake as one through which the pipeline should be able to operate and continue operation after its occurrence, while the larger earthquake should not produce damage that has not been anticipated in the design of the pipeline, structures or facilities. However, to do this in a systematic way usually would involve an unreasonable degree of design effort and moreover often may be based on inaccurate or insufficient seismic data for the region. When this is the case the relationship between the intensities of the two design earthquakes normally has been taken arbitrarily as a factor of two.

The earthquake intensity by itself has limited significance in terms of design to resist seismic motions. Of equal importance are the structural parameters governing response, such as stress or strain and deflection, that the designer intends to use for the particular earthquake hazard selected. Normally these criteria are selected to make the Design Maximum Earthquake govern the design. Furthermore the criteria are such that, in the event of the smaller earthquake, the pipeline and facilities, if properly designed in accordance with the recommendations, will generally be able to continue operation.

Structures and aboveground piping, which acts like a structure, respond to an earthquake in a way associated with their dynamic parameters. Buried piping responds to earthquake motions by moving with the ground in such a way as to have nearly the same curvature and nearly the same longitudinal strain as the ground. Thus, the design criteria for these two conditions must be different. In general, for aboveground elements or structures, the design is carried out by use of the concept of the design or response spectrum. The response spectrum concept, however, does not apply to buried pipe. In the latter case the earthquake motions produce strains in the pipe that can be computed reasonably well from the earthquake ground motions and the estimated wave propagation velocity arising from the earthquake. These differences are explained in more detail later in this paper.

Seismic Design Motions and Response

Actual Versus Effective Earthquake Motions.—Although peak values of ground motion may be assigned to the various magnitudes of earthquakes, especially in the vicinity of the surface expression of a fault or at the epicenter, these motions are in general considerably greater than smaller motions that occur many more times in an earthquake. Design earthquake response spectra are based on "effective" values of the acceleration, velocity and displacement, which occur several times during the earthquake, rather than isolated peak values of instrumental reading. The effective hazards selected for determining design spectra may be as little as one-half the expected isolated peak instrument readings for near earthquakes, ranging up to the latter values for distant earthquakes.

Design response spectra determined from these parameters can take into account the various energy absorption mechanisms, both in the ground and in the element, including radiation of energy into the ground from the responding system.

In assessing the importance of the accelerations and velocities for which the design is to be made, the maximum ground accelerations, in themselves, are of less significance than the accumulated effects of the larger number of somewhat smaller accelerations that contribute to the principal structural or element response. In general the significant effects of an earthquake are measured more directly by the maximum ground velocity than by the maximum ground acceleration. A single spike of high acceleration may have much less significance on response than would be computed by straightforward applications of linear elastic analysis for dynamic systems.

In the design of any system to resist seismic excitation, there are a number of parameters and design considerations that must be taken into account. Among these are the magnitude of the earthquake for which the design is to be made, the distance of the facility from the focus or fault, the parameters governing attenuation of motions with distance from the focus or epicenter, the soil, rock or permafrost conditions as well as the general geologic conditions in the vicinity, and the parameters governing the response of the facility or structure itself. Most, if not all, of these parameters are subject to considerable uncertainty in their value. Because so many of the parameters involved have probabilistic (rather than deterministic) distributions, it is not proper to take each of them with a high degree of conservatism because the resulting combined degree of conservatism would then be unreasonable. At the same time it is desirable to have an assured margin of safety in the combined design conditions. Thus, a choice must be made as to the parameters that will be taken with large margins of safety and those that will be taken with more reasonable values closer to the mean or expected values.

The relation between magnitude of energy release in an earthquake and the maximum ground motion is complex. There are some reasons for inferring that the maximum accelerations are, for example, nearly the same for all magnitudes of relatively shallow earthquake for points near the focus or epicenter. However, for larger magnitudes, the values do not drop off so rapidly with distance from the epicenter, and the duration of shaking is longer. Consequently, the statistical mean or expected values of ground motions show a relationship increasing with magnitude, although not in a linear manner.

Design Seismic Motion.—In selecting the earthquake hazards for use in design, the general concept has been used that the earthquake magnitude selected should be at least as large as those that have occurred in the past, and these earthquakes are generally considered to have equal probabilities of occurring at any point within regions of similar or closely related geologic character. In particular, the estimates of motion considered are appropriate for competent materials at or near the ground surface, including rock and permafrost, or competent consolidated sediments at or near the surface. The values selected are nearly independent of the properties of competent surface materials. It is considered that the predominant part of strong earthquake ground motion, generated by a near shallow earthquake energy release, is represented by surface waves. In general, these are propagated in a manner consistent with the properties of the material at a depth considerably beneath the surface and are not affected to a large extent by the surface properties themselves. The design values of motion are based on the assumption that the same values are applicable in a particular zone for all competent soils.

An example of design seismic motions are those given in Table 1 for four seismic zones for the Trans-Alaska pipeline (11). These zones are characterized by the magnitude of earthquake considered as the Design Maximum Earthquake. For each of the zones two sets of effective ground motion values are listed. The first set, entitled "Ground Motion," includes those values that may affect the stability of slopes or the liquefaction of cohesionless materials, and are also the values which should be used to infer the strains in underground piping. The second set of values, entitled, "Structural Design," lists those values that are to be used for the design of structures or other facilities. These values take into account implicitly the size of the structure, soil-structure interaction, and structural response and are generally less than those used for defining soil instabilities. Obviously the actual values are transient values at variable times, and only the peak effective design values are listed. The design motions given are for the horizontal direction and may occur with equal probability in either of two orthogonal horizontal directions more or less simultaneously.

TABLE 1.—Design Seismic Motions

	Ground Motion			Structural Design		
Magnitude (1)	Acceleration of gravity, as a percentage (2)	Velocity, in inches per second (3)	Displacement, in inches (4)	Acceleration of gravity, as a percentage (5)	Velocity, in inches per second (6)	Displacement, in inches (7)
8.5 and 8	60	29	22	33	16	12
7.5	45	22	16	22	11	8
7.0	30	14	11	15	7	5.5
5.5	12	6	4.5	10	5	4

It is recommended that the design motions to be used in the vertical direction be taken as two-thirds of the value in the horizontal direction.

The maximum ground motion values given in Table 1, as covered earlier herein, may be considerably less than the isolated peak values of motion (as measured by instruments) that correspond to the magnitudes of earthquakes that might be assigned to these various zones.

Elastic Spectral Amplification.—The elastic response of a simple dynamic system subjected to motion of its support is affected to a large extent by the damping in the structure. This damping is usually expressed in terms of the percentage of the "critical value" of damping. Values of damping for particular structures or structural types are covered in Refs. 10 and 14. The importance of damping is indicated by the large effect of damping on the elastic spectral amplification.

The ductility factor of a structure or element is defined as the value of deformation or strain x_m which the structure or element can sustain before failure relative to that value x_y for which it departs appreciably from elastic conditions. It is defined precisely only for an "elastoplastic" relation. However, where the load deformation or stress-strain curve is one which does not have

the characteristics of an initial elastic followed by a perfectly plastic relationship, then the ductility factor must be defined in the fashion given in Refs. 10 or 14 by use of an equivalent elastoplastic relation drawn to make the energy or area under the original curve up to x_y and x_m the same as that under the elastoplastic curve.

The amount of inelastic deformation that a structure can undergo without suffering undue damage also affects its response, in terms of the stresses in it and the corresponding deformations and deflections. The allowable values of ductility depend on the material of which the structure is made and on its manner of construction, principally the way in which joints are made. For example welded steel structures of high quality steel, made with good welding techniques and attention to details possess high ductility. Under certain circumstances the ductility can be impaired by a tendency to fracture brittlely or to undergo local buckling. For these reasons, the ductility levels used in a design must be verified to determine that the materials themselves and their fabrication processes, and especially for reinforced concrete the details of construction, are controlled in such a way that the value of ductility used can actually be achieved while maintaining the requisite margin of strength; it is recommended that the structure as a whole, including details, be made capable of developing a ductility factor of at least 1.5 times that used in the design spectrum. Possible ductility levels under ordinary conditions are covered in Ref. 14. Where the permissible level of structural response does not involve yielding at all, then the ductility factor used is limited to a value of unity.

Response and Design Spectra.—The response spectrum (9,10,13,14) is a plot of the maximum transient response to dynamic motion of a simple dynamic system having viscous damping. An elastic response spectrum has peaks and valleys, but in general has a roughly trapezoidal shape, similar to the upper part of Fig. 1. Spectral amplification factors for horizontal motion, in the elastic range, for damping values of 2%, 3%, 5%, and 7% critical, taken from Ref. 6 (75 percentile values) are given in Table 2.

To draw the elastic response spectrum for any Design Maximum Earthquake motion for a structure, one takes the values of ground motion for any one of the zones from Table 1, using the "structural design" values, and applies the appropriate amplification factors from Table 2 for the particular percentage of damping to the acceleration, velocity, and displacement, respectively. The damping values in Tables 2 and 3 are intended to represent primarily material and structural damping. One obtains in this way a roughly trapezoidal form of response spectrum similar to the curves in Fig. 1. The intersections of the upper two knees of the elastic response spectrum are determined by the amplified motion lines. The two lower knees, at the higher frequencies, are taken as 8 hz and 33 hz, respectively. The value of the spectral acceleration at 33 hz and beyond is taken as the maximum ground acceleration for the elastic response spectra.

Spectra also may be drawn for the operating earthquake for any zone, where the ground motion values are taken as half of those that correspond to the larger earthquake. In general, the amplification values, because of the different values of damping that might be used for the lower intensity earthquake, will not be the same as for the larger earthquake.

To determine the design spectra for acceleration (or seismic coefficient) for

the inelastic case, one takes the appropriate value of ductility factor from Table 3 for the seismic design class (defined later herein) and divides the values of elastic displacement and velocity bounds by the value of ductility factor selected. The values of the controlling elastic acceleration bound, however, are divided by the quantity $(2\mu - 1)^{1/2}$, in which μ is the ductility factor. For frequencies

TABLE 2.—Spectral Amplification Factors, Horizontal, Elastic Range

Damping, per cent critical (1)	Amplification Factor		
	Acceleration (2)	Velocity (3)	Displacement (4)
2	3.4	2.7	2.2
3	2.9	2.4	2.1
5	2.5	2.1	1.8
7	2.2	1.9	1.6

TABLE 3.—Example of Damping and Ductility Levels for Various Design Classes and Earthquakes

Earthquake (1)	Class (2)	Damping, percent critical (3)	Ductility factor (4)
Design probable	I	2	1.5
	II	3	2
	III	5	3
Design maximum	I	3	2
	II	5	3
	III	7	5

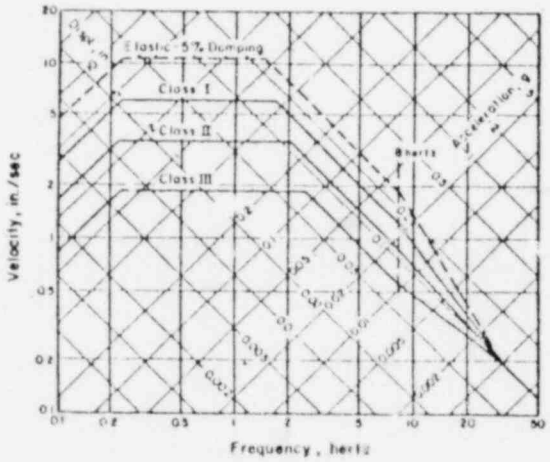

FIG. 1.—Elastic Response Spectrum and Design Spectra

higher than 33 hz, the design acceleration level is the same as the elastic acceleration (10,13). Typical design spectra for the three seismic design classes are shown in Fig. 1.

From the procedure described, it is clear that the intensity of earthquake motion as defined by the applicable response spectrum, must be considered in the light of the way in which that earthquake motion is used in design. In other words, one would prescribe a lower value of acceleration be used with a procedure that involves the use of working stresses than a procedure that involves yield point (or limit) strengths. One cannot compare the earthquake accelerations prescribed by various codes without taking into account the design criteria used in the codes. The Uniform Building Code of the United States, which generally is based on the SEAOC Code, has up to the present time used working stress design criteria, and the seismic coefficients described in the SEAOC Code are consistent with those values. One would have to increase the seismic coefficients in the code to arrive at values comparable with those develop[illegible]in, which are to be used at yield levels.

Classification for Seismic Design

Because of the importance of the amount of deformation or stress that can be permitted in buildings of various types subjected to earthquakes, guidance is necessary in arriving at an appropriate means of selecting the design requirements. For this purpose, a seismic classification system, encompassing three classes, is recommended for use.

Class I includes those items of equipment (including instruments) performing vital functions that must remain nearly elastic, or any items for which the allowable probability of exceeding design levels must be extremely low. Obviously, items that are essential for the safe operation of the pipeline or any facility thereof, where damage to the particular unit would cause extensive loss of life or major environmental damage, would be in Class I. Other items might be included in Class I if failure of such items would entail large costs in repair or replacement, or lengthy shutdown of the pipeline.

Class II includes aboveground piping or buildings and equipment that can deform inelastically to a moderate extent without loss of function. This class also includes any items for which the allowable probability of exceeding design limits can be somewhat larger than in Class I. However, piping which might fail in a brittle mode, or whose failure might tend to propagate over considerable distances, causing extensive damage or possibly danger to life in populated regions, or both, perhaps should be put in Class I or in a classification intermediate between Classes I and II.

Class III includes, in general, buildings or equipment that can be permitted to deform significantly, or any items that are not essential for safety; it includes those items for which the allowable probability of exceeding design limits can be moderately high. However, buildings that contain Class I or Class II items and which might damage or put out of action those items if the Class III buildings should deform excessively, should be moved to a higher class, perhaps intermediate between Classes II and III.

An example of the damping and ductility factors that might be used in defining the design spectra for the various seismic design classes is given in Table 3.

These give results that are consistent with the class definitions above and satisfy the criterion that the Design Maximum Earthquake, with its higher intensity, should in general give more stringent requirements than the Design Probable Earthquake.

Response of Pipelines and Structures

The response of a structure or element is dependent on its strength, damping characteristics, and the stress-strain or load-deformation relationship for the structure or element considered. The response also is affected to a large degree by soil-structure interaction in those instances where the structure is supported in or on soil, and not on rock. In particular, the following assumptions normally are made.

Belowground or buried pipe, as depicted in Figs. 2(*a*) and 2(*b*) is considered to move with the ground in such a way as to have nearly the same longitudinal strain as the ground. These requirements impose both compressive and tensile forces in the pipe as well as lateral bending. Of course, this assumption is

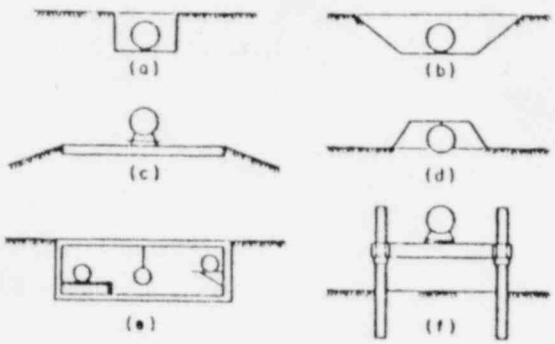

FIG. 2.—Piping Configurations

valid only so long as the material surrounding the pipe remains relatively intact. In other words, the assumption applies only if the material surrounding the pipe does not liquefy or the material surrounding the pipe is not grossly disturbed. Under liquefaction conditions, the pipe is no longer supported directly by the material and the possibility of further large deformations must be considered.

For aboveground pipelines [Fig. 2(*f*)] the motion of the ground is imparted to the pipe through the piers or supports under the pipe. The deformation of these supports must be considered, as must also tipping or tilting. Of course, liquefaction of the foundation material under the support can mean loss of that support.

Consideration also must be given, both for buried and aboveground pipelines, to the relative motions arising from faults crossing the pipeline. Vertical and horizontal displacements of several feet might occur where fault movements take place. But these need not necessarily cause rupture or failure of the pipe [illegible]

failure by a properly designed and supported aboveground pipeline [Fig. 2(*f*)], even though one or two supports may lose contact with the pipe. For underground pipe such motions might cause severe local distortion and wrinkling but not necessarily collapse or rupture.

One of the most important response parameters is the level of permissible response. For some structures the response must remain in the elastic range. This is normally not the case with any element of a pipeline, for large deformations generally can be permitted to occur provided rupture does not take place with a consequent hazard to the public or to the environment. However, it is important to recognize the fact that the level of response must be selected in a manner consistent with the selection of the earthquake itself. In order to reach a consistently reasonable margin of safety, it is suggested that the level of response permissible in the pipeline under extreme conditions, i.e., for the maximum magnitude of earthquake and the maximum intensity of motion, involve a considerable degree of deformation but short of rupture of the pipe.

Yielding and Wrinkling as Function of Allowable Pipe Deformation.—Aboveground piping and structures containing resisting elements composed of piping normally would fall in Class II or Class III depending on their importance and influence on safety. The methods of analysis and the design spectra used are the same as for structures having similar or related properties. However, in piping one must take account of the stress considerations (including long-term corrosion potential) at joints or connections or points of support to insure that the ductility levels consistent with the design classification can be met. These generally can be met by piping in Class II, but in Class III local strengthening of the pipe may be required if advantage is desired to be taken of the lower design acceleration values for that class.

Stresses in the materials of either pipe or other structural elements, for earthquake and primary stresses combined, should be limited to minimum specified yield strength values in general. However, for the Design Maximum Earthquake, the values might be increased to the average actual yield point. Larger values of deformation might be permissible in extremely ductile structures depending upon the nature of the pipeline, contents handled, environmental concern and safety required but the limiting values applicable for Class III should not be exceeded. Development of applicable stress and deformation criteria to accommodate the seismic criteria outlined, as well as applicable code provisions (1,2) for example, normally entails considerable effort by the designer and often involves review by cognizant governmental regulatory agencies.

Local wrinkling theoretically (17) may begin at compressive strains given by the following expression

$$\epsilon = 0.6\,\frac{t}{R} \qquad (1)$$

in which t is the wall thickness and R is the pipe radius, both in the same units. Actual pipe normally will begin to wrinkle at strains one-third to one-quarter that value. Strains of the order of 4 to 6 times as great, can be sustained without danger of tearing at the compression wrinkle. Because of the shift of the neutral axis away from the compressive side when wrinkling occurs, the tensile strains will be considerably less than the compressive strains under

these conditions. If the stresses causing wrinkling arise in part from thermal effects or other secondary sources, which is usually the case, the likelihood of failure is reduced even more.

Design Criteria and Procedures for Structures and Aboveground Pipelines

Design Considerations.—For the design classifications used, Class III is considered as falling under the provisions of extant codes for ordinary buildings. Thus, the concept is implicit in the recommendations made herein that Class III items should not have design levels lower than those for the applicable codes, such as Ref. 15. Normally it would be expected that major structures and aboveground piping will be placed in Class II, except under circumstances where buildings, piping, or equipment can be permitted to deform a great deal, or are not essential for safety, or will not seriously damage any elements or items that are essential for safety.

After selecting the classification, the design spectra can be drawn by use of structural design motions of the type given in Table 1, and the amplification, and damping and ductility values like those of Tables 2 and 3. The design coefficients are then determined. One may choose to use the simple methods of analysis prescribed in the various building codes. In the event that a dynamic analysis is made it is recommended that the response spectrum technique be used. The various modes of response are computed, and then for each modal frequency the spectrum amplification factors are read from a plot similar to that of Fig. 1. The various modal values for stress, deformation, or other response at a particular point are then combined for the various modes by taking the square root of the sums of the squares of the individual modal responses.

The dynamic analysis procedure should generally be used for complex or unusual structures, but it is quite adequate in many cases to use simplified code procedures with the appropriate seismic coefficients obtained from the response spectra constructed by the procedure described in.

Attention is called to the fact that the design spectra, Fig. 1 for Class I, Class II, and Class III can be used only to obtain acceleration levels or seismic coefficients but not deflections or deformations. In order to obtain displacements or deflections, one must multiply the design spectra by the appropriate value of ductility factor, such as that given in Table 3. In general, this will lead to displacements that are equal to or greater than the elastic spectral displacements in all cases. For frequencies higher than about 2 hz, the total displacements are slightly to considerably greater than the corresponding elastic displacements, but for lower frequencies, they are precisely the same.

Combining Horizontal and Vertical Seismic Motions.—For those parts of structures or components that are affected by motions in various directions, the net response may be computed by either one of two methods. The first method involves computing the responses in a particular direction at a particular point for each of the directions independently and then taking the square root of the sum of the squares of the resulting responses as the combined response. Alternatively, one can use the procedure of taking 100% of the motion in one direction, combined with 40% of the motions in the other two orthogonal directions, then adding the absolute values of the resulting responses to obtain the maximum combined value in a member or at a point in a particular direction.

In general, this alternative is slightly conservative for most cases and is quite adequate since its degree of conservatism is relatively small.

Gravity Loads.—The effects of gravity loads, when structures deform laterally by a considerable amount, can be of importance. In accordance with the general recommendations of most extant codes, the effects of gravity loads are to be added directly to the primary and earthquake effects. In general, in computing the effect of gravity loads, one must take into account the actual deflection and not that corresponding to the reduced seismic coefficient. In other words if one designs for one-fifth of the actual acceleration, as one does when using seismic Class III, the actual total lateral deflections of the structure are obtained by multiplying the elastically computed deflections for the design accelerations by five.

Unsymmetrical Structures and Torsion.—Consideration should be given to the effects of torsion on unsymmetrical structures, and even on symmetrical structures where torsion may arise accidentally, because of various reasons, including lack of homogeneity of the structures, or the phased wave motions developed in earthquakes. The accidental eccentricities of the horizontal forces prescribed by current codes require that 5% of the width of the structure in the direction of the earthquake motion considered be used as an accidental eccentricity. The stresses arising from the actual eccentricity should be combined with those arising from the accidental eccentricity in all cases. The effect of eccentricity is to produce a greater stress on one side of the structure than on the other, and the outer walls and columns will in general be subjected to larger deformations and forces than would be the case if the structure were considered to deform uniformly.

Overturning and Moment and Shear Distribution.—In general when modal analysis techniques are not used, in a complex structure or in one having several degrees-of-freedom, it is necessary to have a method of defining the seismic design forces at each mass point of the structure in order to be able to compute the shears and moments to be used for design throughout the structure. The method described in the SEAOC Code (15) is preferable for this purpose.

Design Criteria and Procedures for Buried Pipeline

For buried piping, the pipe in general will deform with the ground, and the strain in the ground will be transmitted to the piping without attenuation. Where faults intercept the pipeline, and the motion is greater than that which can be absorbed by change in cross sections of the pipe itself, the structural provisions must be made to prevent the pipe from rupturing because of the fault motion. These considerations are covered in the following.

In general, earthquake motions on buried pipe produce essentially secondary, rather than primary, effects since the strains or deformations are fixed in amount and the size of the pipe or thickness of the wall, or quality of material, does not affect the strains appreciably. The implications of yielding and wrinkling in the pipe caused by ground motions, as covered earlier, may have an importance different from the effect of secondary stresses on aboveground structures.

Strains In Buried Pipe.—Because a buried pipe conforms to the strains and deformations in the medium in which it is placed, both longitudinal strain and curvature are induced in a buried pipeline. Actually, because of slip between

the pipe and the medium, and local deformations between the two, including some slight ovalling of the pipe, the deformations of the pipe may be slightly less than that of the medium. It is generally not desirable to consider a reduction from the strain in the medium for the above reason however. One can make inferrences about the relative motions between nearby points in a pipe as outlined here (3). For example, consider two points at a distance b apart, and consider a displacement ρ at point 1 and ρ plus an increment at point 2, in the same direction as at point 1. If a wave is propagated from point 1 towards point 2, with a displacement of the form given by

$$\rho = f(x - ct) \quad (2)$$

in which c is the velocity of this particular wave propagation and t is the time, then the various derivatives of the displacement ρ with respect to x and t are given by the following relations (4):

$$\frac{\partial \rho}{\partial x} = f'(x - ct); \quad \frac{\partial^2 \rho}{\partial x^2} = f''(x - ct) \quad (3)$$

$$\frac{\partial \rho}{\partial t} = -cf'(x - ct); \quad \frac{\partial^2 \rho}{\partial t^2} = c^2 f''(x - ct) \quad (4)$$

From the first of Eqs. 3 and 4 one derives the following result:

$$\frac{\partial \rho}{\partial x} = \frac{1}{c}\frac{\partial \rho}{\partial t} \quad (5)$$

and similarly, from the second of Eqs. 3 and 4 one obtains

$$\frac{\partial^2 \rho}{\partial x^2} = \frac{1}{c^2}\frac{\partial^2 \rho}{\partial t^2} \quad (6)$$

In the case where ρ is in the direction of x, then the strain ϵ is obtained from Eq. 5, and the maximum strain at point 1 is therefore

$$\epsilon_m = -\frac{v_m}{c} \quad (7)$$

In the case where ρ is perpendicular to x, either horizontally or vertically, the maximum curvature at point 1 is obtained from Eq. 6, and is as follows:

$$\text{Curvature} = \frac{a_m}{c^2} \quad (8)$$

in which a_m is the maximum acceleration at point 1. The strains corresponding to such effects are commonly quite small.

When the displacements in the region considered are associated with horizontal shearing displacements occurring without longitudinal or extensional strain, then the displacement ρ is perpendicular to the wave front. For this case there is also an extensional deformation of an embedded element such as a pipe or tunnel, but the relations governing it are slightly different from the case where ρ is normal to x, but the element considered makes an angle

θ with x, the shearing distortion γ in the element is $\gamma = (v/c)\cos^2\theta$ and the maximum value is given by

$$\gamma_m = \frac{v_m}{c} \quad (9)$$

However, for the maximum longitudinal strain in the element $\epsilon = (v/c)\sin\theta\cos\theta = (1/2)(v/c)\sin 2\theta$ and the maximum value is given by

$$\epsilon_m = v_{m/2c} \quad (10)$$

For either Eq. 7 or 10, slippage of the soil against the element may reduce the force transmitted to it from that corresponding to the strains determined from the equations.

In applying the preceding expressions the value of wave propagation velocity, c, to be used in arriving at the pipe strain or curvature is the effective velocity applicable to the type of motion and medium being considered. In the case of shear wave effects, which is typical, the effective value normally should not be taken as the value at the surface nor the value at great depth in underlying strata, but instead as the value representative of the actual motion of the medium surrounding the point of interest; in general the propagation takes place in a manner represented in Section 5 of Ref. 8. Examples of effective velocity for shearing type propagation in different media that might be representative under special circumstances are 4,000 fps for rock or permafrost, 3,500 fps for massive gravel deposits, 3,000 fps for sand and competent soils, and slightly lesser values for silt and clay deposits. Obviously, significant relative motions can occur at ground medium transition zones (e.g., rock to soil) and these situation must be carefully considered in design.

On a gross basis considering relative settlement a strain in the pipe of the order of 0.004 has been a common operating limit of deformation; for a 4-ft diam pipe, for example, this corresponds to a radius of curvature of the pipe of the order of 500 ft due to relative settlement. This would correspond to about a 1-in. settlement over a length of 20 ft, or a 2-ft settlement over a length of 100 ft.

A reasonable criterion for permissible deformation to avoid rupture appears to be of the order of 1% to 2% strain in modern steel pipe at any section, computed on a nominal basis, or approximately twice as much at points of stress concentration, such as near welds or abrupt cross-sectional changes, taking into account the local strain or stress concentrations. To reach a strain of the order of 0.01 (corresponding to a ductility factor of about 5) would correspond to a radius 0.4 as large as those just cited which would give relative displacements of the order of about 2.5 times those computed.

Yielding of Buried Pipe.—Because a buried pipe is subjected essentially to longitudinal strain that is fixed in amount, it can be considered as having a higher factor of safety against such strain than it would have against the corresponding stress generated by such a strain applied as a primary loading. It is desirable to keep the tensile strains low to avoid initiation of a brittle fracture in pipe that does not have the assured ductility, especially if it must operate at low temperatures. The figures previously quoted for allowable tensile strain assume that the material qualities as measured by transition temperature

or in appropriate other ways (e.g., fracture mechanics concepts) will insure against brittle or ductile fracture at these levels of strain. The permissible strain levels would have to be reduced in the event such assurance is not possible.

Although brittle fractures are not likely to occur under compressive strains in the pipe, wrinkling or buckling can occur. In general, current piping codes require that the strain in piping that is constrained be limited to somewhat less than the minimum specified yield strength. This is probably desirable from the point of view of operating limits since buckling may produce difficulties in operation, but insofar as allowable maxima are concerned, it does not appear necessary to limit the compressive strain even for buried piping to values less than about 1% to 2% strain. This, however, should be the sum of the strains at a point in a given direction from all sources, including thermal, pressure, and seismic deformation. Particular care needs to be exercised at bends, either side bends, over bends, or sag bends, however, to avoid buckling or compressive or tensile failures that arise from the combined longitudinal stress and moment. In some cases where special provision must be made for deformation of belowground piping, the mounting of the piping in tunnels with special supports, as shown in Fig. 2(e) may be worthy of consideration; normally however this technique is quite expensive.

Special Geotechnical Design Provisions

Stability and Dynamic Movements of Slopes.—The movement of slopes and embankments under seismic conditions is covered in Ref. 3. For equal resistances to sliding in both directions the maximum motion is given by the relation:

$$\frac{V^2}{2gN}\left(1-\frac{N}{A}\right) \qquad (11)$$

in which V is the maximum ground velocity in the earthquake, g is the acceleration of gravity, N is a measure of the dynamic resistance to sliding (as determined from the constant horizontal or inclined force which, if applied after several cycles of shaking and consequent loss of strength, would produce sliding with a factor of safety of unity), and A is the maximum ground motion acceleration in the earthquake considered. The most serious case is that in which slip takes place in only one direction, corresponding to an unsymmetrical resistance to motion. The maximum displacement under earthquake loading, as computed from the extreme values for a number of calculations from different earthquakes, is approximated by the relation:

$$\frac{V^2}{2gN}\left(1-\frac{N}{A}\right)\frac{A}{N} \qquad (12)$$

The values given by Eq. 12 becomes highly overconservative when N/A is less than about 0.15. In this case, Eq. 12 should be replaced by an upper bound of:

$$\frac{6V^2}{2gN} \qquad (13)$$

Eqs. 12 and 13 are based on rigid-plastic resistance to sliding, and give good

estimates of the maximum permanent displacement after the excitation has stopped. In general, the transient displacement does not exceed about twice the maximum permanent displacement, but may be considerably less, especially when the permanent displacements are large.

In studies of slope stability, the value of N is determined by taking it as generally equal to that constant acceleration of the earthquake applied as a constant dynamic force which gives a factor of safety of one for the slope. If one-half the maximum ground motion acceleration is taken as N, or if the slope has a factor of safety of one with 1/2 A statically applied, one can determine from Eq. 12 the maximum downslope displacement, and normally this will be a few inches in magnitude.

It should be pointed out that most embankments and especially most earth and rock fill dams are designed for a considerably smaller factor of safety than that which would correspond to a value of dynamic factor of safety of 1.0 as previously defined. Values of displacements of several feet are quite possible in well-designed dams and dikes without failure occurring, and have been experienced in practice. In any event, if the ratio of N/A is greater than about 0.5, one does not ordinarily consider that failure of an embankment will occur. Where the values are less than this, then the embankment design must be considered more carefully and in detail. The same comments apply to dikes and similar structures, including gravel pads or berms.

It is estimated that slips of one foot or less for a slide of normal dimension would cause plastic deformation but not cracking of standard dimension metal buried pipe; pipe made of brittle material normally could not withstand such motion.

Where slopes are unstable and on the verge of sliding statically, the use of Eq. 12 would indicate that large displacements would occur. Such regions should be avoided or, as an alternative, contingency plans should be developed. Obviously where exceptional cases are encountered special engineering considerations may be necessary.

Liquefaction Potential.—One of the most serious consequences of an earthquake is the effect of changing the properties of inundated sands or cohesionless materials so that they become "quick" or develop a liquefied condition. One method of dealing with this problem is covered in Ref. 16. However, it must be noted that several peaks of high acceleration have little influence compared with that of long-sustained motions, even of a less intense nature; the latter can have a serious effect on the liquefaction potential. For this reason, it is expected that high intensities of only 2 or 3 spikes of acceleration can be discounted and the liquefaction effects can be computed for longer durations of shaking with a corresponding lower acceleration level not less than the design ground motion value such as those presented in Table 1. This procedure also takes account of the high damping values expected.

Effect of Fault Motions.—The transient displacements at some distance away from a fault are not negligible but are not sufficiently large to cause difficulties, especially since they do not correspond to large relative displacement. A procedure for estimating fault motions is presented in Ref. 12. However, at or adjacent to a fault, large relative motions can occur. In rock, these relative motions take place over fairly short distances, and therefore for buried pipe provisions must be made to cushion the pipe against the abrupt changes in displacement.

This can be done most readily by arranging for the excavation of the trench in which the pipe is placed to have relatively shallow (45° or less) side slopes, with a limited depth (not more than 3 ft-5 ft) of gravel cover over the pipe [Fig. 2(*b*)], so that the pipe will be lifted up and out rather than crushed and cracked if a fault occurs transverse to the pipe. With nearly vertical slopes [Fig. 2(*a*)], the motions would tend to cut or constrict the pipe in a way that would cause danger of overstraining and possibly failure. Fault motions in soil are not nearly so serious as they are in rock because they do not occur so abruptly. In soil they would be expected to correspond to more gradual displacements in which the local pressures against the pipe would cause some deformation of the pipe but not the kind of crushing or damage that would be caused by faulting in rock; in some cases it may be desirable to place the pipe on the surface [Fig. 2(*c*)] or in a berm on the surface Fig. 2(*d*).

Vertical fault motions are not generally serious for metal pipe since the pipe normally has the capability to resist a 4-ft or 5-ft vertical displacement without undue difficulty, corresponding to loss of one or two of the aboveground supports; if it can become free it can accommodate significant motions over large distances.

Where fault motions might occur, it is important that the depth of cover over the pipe be limited, as explained above, and that anchors or bends of any sort not be placed within a distance of at least 200 ft either side of the expected fault area. It is also desirable that the pipeline intersect the fault at nearly right angles, or in any case not make an angle less than about 45° with the fault trace. If the geometry and direction of relative slip are known it may be desirable to incline the pipe axis to the fault slightly to produce either tension or compression in the pipe depending upon the design criteria and to accommodate such items as the type of pipe material, and over bends or sag bends. Also, long-term creep effects near faults may need consideration in design.

Acknowledgment

The criteria and procedures described have been developed by the writers over some period of years in connection with design studies for the Trans-Alaska pipeline (7,11), the Canadian Arctic gasline, the Sohio pipeline from Long Beach, Calif. to Midland, Tex., as well as for other special facilities. The recommendations made herein are not to be construed as representing an official position pertaining to any of the projects identified.

Appendix I.—References

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