Transactions, **SMiRT-22**
San Francisco, California, USA - August 18-23, 2013
Division VI: Paper ID# 283

EVALUATION OF NUCLEAR SAFETY RELATED BURIED PIPES FOR SURFACE IMPACT LOAD

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ABSTRACT

The evaluation of a buried pipeline for surface impact loading can be performed using any one of a number of methodologies. The range of solution approaches to the buried piping surface impact response problem is indeed broad and there is no literature currently available which provides guidance for comparison and selection of the various methodologies, with consideration of their applicability and conservatism. Therefore, it is often left to the engineer to determine a suitable approach. Lacking clear guidance, conservative assumptions are often resorted to. From a construction perspective, relaxation of these conservatisms may provide significant economic returns. For example, the outage duration for conventional nuclear plant maintenance activities is significantly influenced by the constraints on construction lift paths and constraints on load lift magnitudes, which are in turn a function of the capacity of underground utilities to withstand postulated dropped loads. Should such conservatism be relaxed, scheduling may be improved as a result of increased construction flexibility. The purpose of this paper is to document and compare published solutions for buried pipe structural responses to surface impact.

The free-field soil stress response can be determined by finite element analysis, by differential equation analytical methods using integral transforms, by scaled energy data collected from field measurements of soil response to pile driving and deep soil compaction activities, or by hand calculations using energy balance or conservation of momentum principles. These various methods are summarized and recommendations are made. Further assessments are made of soil-structure interaction effects (versus decoupling the structure from soil and application of the free-field soil strain or stress for evaluation of the pipe).

INTRODUCTION

In a nuclear power plant, safety related buried pipelines need to be assessed for the effects of postulated surface impact loads such as tornado missile impact and accidental load drop. Examples of such pipelines include emergency firewater lines and service water lines. Various approaches have been published concerning such evaluations. The simplest approach is to determine the surface impact loading, and to impose the resulting analytically or empirically estimated free-field soil strain onto the pipeline. Alternatively, hand calculation methodologies are available which provide some approximate account of pipe-soil interaction effects, which may be important for larger diameter, shallow buried pipelines. Finite element (FE) pipe-soil dynamic interaction response solutions, using computer programs such as LSDYNA or SASSI, are also available. However, there is no literature currently available for comparison and selection of the various methodologies. The purpose of this paper is to document and qualitatively compare published solutions for surface impact loads, in terms of applicability and conservatism, in order to provide a clear and easy-to-use guidance for such evaluations.

Prior to performing evaluations based on either detailed closed form analytical type solutions (integral transform methodologies) or performing a finite element solution, one or more of the approximate simpler methodologies should always be implemented. These hand calculation approaches fall into two classes, 1) those using formulas developed from strength of materials, conservation of energy and conservation of momentum principles, and 2) those based on field measurements of wave attenuation characteristics observed during pile driving, blasting, or soil compaction activities. Depending on the soil/buried-structure system characteristics, these simplified methodologies may or may not be sufficient by themselves to support nuclear plant safety-related design. Where more detailed evaluations are indicated, the simpler methods serve as order-of-magnitude benchmarks. Two major sources of error are inherent in the buried structure's response when such methods are used: error in the estimate of the free field velocity and error in the structural response calculation resulting from decoupling the structure from the soil.

Analytical methods for the evaluation of buried pipes subject to surface impact loads utilize the dynamics of the elastic soil continuum. Given that these methods are aimed at closed-form solutions for the dynamic response of the soil media using wave propagation theory, the theoretical models are quite complex and are based on specific sets of assumptions and simplifications. These solution techniques still have limited applicability to the determination of response of buried pipes, since they do not account for the disturbance of the free-field stress profile due to the presence of the pipes. Nonetheless, conclusions gained from the application of these analytical techniques to simpler, easy to comprehend problems provide invaluable insights into the broader understanding of the underlying phenomena in addition to helping validate solutions based on hand, empirical and finite element methods.

DETERMINATION OF SURFACE IMPULSE

Consider an object of weight, W, falling from a height, h, above the ground surface with a striking velocity of V_0 . The change of momentum for the falling object results from a starting velocity decreased to zero. Assuming a single degree of freedom spring mass system for the soil and an impulse loading (forcing function) of triangular shape as shown in Figure 1, the maximum surface impact force, F_{max} , and the impact duration, t_d , may be obtained using principles of conservation of momentum (Mayne and Jones, 1983).

$$F_{max} = \sqrt{\frac{32hGr_0W}{\pi^2(1-\nu)}} \quad , \quad t_d = \pi\sqrt{\frac{W(1-\nu)}{4gGr_0}} \tag{1}$$

where the object mass is W/g, the vertical soil stiffness is $4Gr_0/(1-\nu)$, $r_0 = L/\sqrt{\pi}$ is the equivalent radius for the impact area, L is the maximum dimension of the falling weight, and G and ν denote the shear modulus and Poisson's ratio of soil, respectively. For large soil strain response, the shear modulus is about one-tenth of the low strain shear modulus as determined from geophysical tests. Hence, $G = \rho {V_s}^2/10$, where ρ is the soil density and V_s denotes the low-strain shear wave velocity in the soil medium. It is noted that the shear modulus degradation rate varies considerably for different types of soils and decreases nonlinearly with increasing shear strain in the soil (EPRI, 1993). This may be considered as needed to determine the shear modulus compatible with the applicable strains in the soil medium surrounding the buried pipeline.

Alternatively, conservation of energy may be used in lieu of conservation of momentum to calculate F_{max} and t_d .

$$F_{max} = \frac{2Wgh}{gx_{max}} \quad , \quad t_d = \frac{2x_{max}}{V_0} \tag{2}$$

The penetration depth into the soil, x_{max} , of the falling object has been studied extensively. One well accepted empirical result, the Petry formula (Amirikian, 1950; ALA, 20017) is given by

$$x_{max} = k\frac{W}{A}\log_{10}\left(1+\frac{{V_0}^2}{215000}\right) \tag{3}$$

where x_{max} is the penetration in feet; k is an experimentally determined coefficient for which values are given in Table 1, A denotes the impact area in ft^2; W and V_0 are in lb and ft/s, respectively.

Table 1: Coefficient k

Soil type	k value
Sandy soil	0.0367
Soil with vegetation	0.0482
Soft soil	0.0732

An alternative empirical equation to calculate x_{max} proposed by Young (1969) is

$$x_{max} = \begin{cases} 0.53\sqrt{W/A}\,\ln(1+2{V_0}^2/10^5) & \text{for} \quad V_0 \le 200 \text{ ft/s} \\ 0.0031SN\sqrt{W/A}\,(V_0-100) & \text{for} \quad V_0 > 200 \text{ ft/s} \end{cases} \tag{4}$$

where S and N are respectively soil constant and missile nose performance coefficients given by Young (1969); A denotes the impact area in ft^2, W and V_0 are in lb and ft/s, respective.

A third method that can be used for determination of surface impulse is to solve the transient contact problem using finite element software such as LSDYNA. The method, however, is rarely used to merely determine the impulse. But when an FE analysis may be required for other reasons, this method provides a better definition of the impulse.

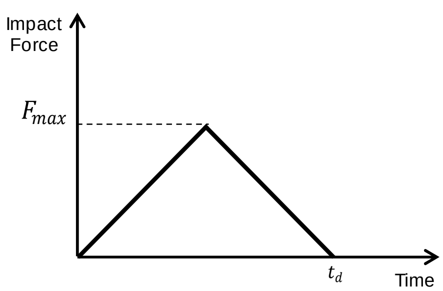

Figure 1. Triangular Forcing Function

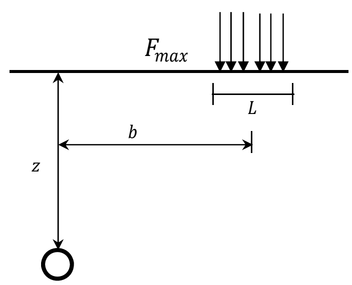

Figure 2. Boussinesq's Equation to Determine Vertical Pressure at Pipe Crown

HAND METHOD TO EVALUATE PIPE RESPONSE

The hand calculation approaches can determine the free-field soil pressures, at various depths below the grade, due to a heavy object dropped from some height above grade. Various hand methods have been developed in order to evaluate responses of pipes which are located beneath or at short distance (horizontally) from the surface impulse. For these pressures which are calculated for locations directly below the impact, their application to off-axis locations within the soil are conservative. The most useful published hand calculation methods are those of Mayne and Jones (1983), Scott and Pearce (1975), and various versions of Boussinesq's equation (ALA, 2001; Amirikian, 1950; Gupta and Saigal, 2003; Das Braja, 1985). Scott and Pearce's work (1975) models the falling-weight/soil system as an single degree of freedom (SDOF) oscillator, with the soil stiffness developed from the vertical, one-dimensional compression wave propagation characteristics of the soil. Formulae are derived for the contact interface force for various states of soil compaction and saturation. While the above simplified hand calculation approaches are obviously deficient due to their one-dimensionality, they can give good results for on-axis drops above shallow buried structures.

The first step of the evaluation using a hand method is to determine the consequential maximum vertical pressure, q_{max}, directly on the top of the pipeline, resulting from the maximum surface impact force F_{max}. Boussinesq's equation embraced in relevant design guidance such as ALA (2001) is widely accepted for this purpose:

$$q_{max} = \frac{3F_{max}}{2\pi z^2 \left[1 + \left(\frac{b}{z}\right)\right]^{2.5}} \tag{5}$$

in which b is the horizontal distance from the impact location to the pipeline axisand z represents the depth beneath impact surface as shown in Figure 2.

Taking into account the foot print dimension of the impulse, Gupta and Saigal (2003) proposed a revised Boussinesq formula as

$$q_{max} = \frac{F_{max}}{\pi BL}[(\beta_2 - \beta_1) + \sin\beta_2\cos\beta_2 - \sin\beta_1\cos\beta_1] \tag{6}$$

in which, $\tan\beta_1 = (b - 0.5L)/z$ and $\tan\beta_2 = (b + 0.5L)/z$ respectively; and L is the width of the falling object

An alternative Boussinesq's equation considering the foot print dimensions of the impulse as $B \times L$ is given by Das Braja (1985) as:

$$q_{max} = \frac{F_{max}}{BL}\frac{1}{4\pi}\left[\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1} \times \frac{m^2+n^2+2}{m^2+n^2+1} + \tan^{-1}\left(\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2-m^2n^2+1}\right)\right] \tag{7}$$

where, $m = B/z, n = L/z$

The empirical solution proposed by Mayne and Jones (1983) is available in lieu of Boussinesq's equation:

$$q_{max} = \frac{V_s\sqrt{WhL}}{(L+z)^2} \tag{8}$$

The units in Eq. 8 are in lb, ft and sec. Peak vertical pressure predicted by Eq. 8 have been confirmed in Mayne and Jones (1983) with field measurements for various soils. Note that Eqs (7) and (8) are for on-axis drop case only. A general Boussinesq formula for point load on an elastic half space and response at any subsurface location is provided in Kachanov et al (2003

Dynamics amplification of the soil response is not included in Eqs. 5 to 8 for determination of q_{max}. This is justified based on pipe-soil interaction studies which are discussed in another section in this paper. In practice, a dynamic load factor between 1.0~1.2 may be employed to ensure conservatism based on engineering judgment.

The analysis of the pipe section for calculated vertical pressure, q_{max}, may be performed using Spangler's Iowa formula (ALA, 2001; Moser, 2001).

EVALUATION OF BURIED PIPES USING MEASURED FIELD DATA

The methodology for evaluation of buried structures using measured field data is to estimate the free-field stress wave amplitude resulting from the surface impact loading (in the form of peak particle velocities) and, neglecting dynamic soil-structure interaction, to apply these free field responses to the buried pipeline and determine the structural response using methods such as those available in Yeh (1974) and Hashash et al. (2001). For this purpose, nuclear plant structural engineers have utilized vibration attenuation data collected for various impact phenomena, such as pile driving, construction equipment vibrations, blasting, soil compaction, etc. The data are typically presented in the form of soil surface peak particle velocities (ppv) as a function of the distance from the impact or as ppv vs. scaled energy plots (Wiss, 1981; Lukas, 1980; Woods and Jedele, 1985). Data for measurements beneath the surface are rare, first because the purpose of this data is generally for use in evaluating the potential for damage to surface mounted structures such as buildings, and secondly because of the difficulty and expense of acquiring reliable undisturbed in-situ data below the surface. Therefore, conservatively, surface free-field motions are used to evaluate sub-surface buried structures (soil motions at horizontal distance from a surface impact are dominated by the Raleigh wave, the amplitude of which attenuates rapidly with depth).

The data are usually represented as straight lines on log-log plots. In the design application of these data to off-axis sub-surface impacts, either the surface ppv directly above the point of interest is used (on the basis that the Rayleigh wave dominates the response and the Rayleigh wave response is maximum slightly below the surface) or the distance from the impact point on the surface to the sub-surface location is calculated and this distance can be used to calculate the "scaled distance" (scaled distance is defined as $[d/E^{1/2}]$,where E is the drop energy and d is the horizontal distance from source to receiver) for determination of the ppv value. A more appropriate design application of the near surface ppv data to off-axis sub-surface locations, taking into account the depth of the pipe beneath the surface, has been developed in Johnson et al (2007) based on extrapolation in accordance with the theoretical half-space transient response solutions. Once the free-field peak particle velocities have been determined at the buried structure location, the corresponding strains can be imposed on the structure, utilizing the conventional relations for the seismic response of buried structures (Hashash et al., 2001). In addition, for open sections, such as pipelines, the ovaling or cross-sectional distortion response is also evaluated by pseudo-static application of the free-field ground loading and evaluation of buried structure for static loads (ALA, 2001; Moser, 2001). The scaled energy approach may result in erroneous responses when SSI effects are significant. The response of a large diameter, thin walled pipe located near the ground surface and subjected to a surface impact loading several diameters away could be significantly influenced by SSI effects and wave diffraction

ANALYTICAL DETERMINATION OF FREE-FIELD RESPONSE

The disturbance of the elastic half space due to an impulse loading applied at the surface can be represented analytically using a system of partial differential equations which represent the three-dimensional propagation of the various wave types in the elastic medium. The differential equation solution for practical problems is cumbersome since it involves the numerical inversion of integral transforms, which which in turn requires numerical complex integration with considerations of limits and continuity at singular points. Such solutions have been published in the literature, but only for special and limiting loadings (step functions) and for specific response locations (surface or on-axis). The propagation of transient stress waves produced in an elastic half-space by sudden application of a constant force was partially solved by Eason (1966) using integral transform techniques. Eason's work, which was an extension of work done by Pekeris (1955), presented the displacement solution at any interior point of the half-space in a form suitable for numerical evaluation, while earlier authors had limited their solutions to surface response only (Pekeris, 1955). Mooney (1974) extended the work done by Pekeris to investigate the influence of source waveform shape and Poisson's ratio of the elastic half-space. Further, Laturelle [1990, 1991] extended Eason's work to provide stress propagation expressions valid in the entire elastic half space for the case of sudden application of pressure over a circular area and provided solutions to account for the effects of surface pulse durations. More recently, Guan and Novak (1994) and Park and Kausel (2004) have presented solutions using Green functions.

Besides the complex nature of the available analytical techniques and difficulty associated with actual implementation to engineering problems, the simplifying assumptions required to achieve a solution, limit the suitability of these results to design applications. Some authors have developed useful graphical non-dimensional solution results (Laturelle, 1990; Guan and Novak, 1994; Georgiadis et al., 1999); however, these solutions still have limited application to the evaluations of buried pipes due to their restricted ranges on system parameters

FINITE ELEMENT APPROACHES

The finite element method has been used by a few investigators to evaluate underground pipelines for the effects of postulated surface impact loads. The method has been used both to calculate free-field responses and for response including the effects of soil-pipe interaction. Two dimensional (2D) finite difference models were used by Schroder and Scott (2000) to study the interaction of elastic waves with a buried land mine. Gupta and Saigal (2003) used ABAQUS to validate the proposed Boussinesq's equation as shown in Eq. (6). An axisymmetric model of ANSYS was used by Johnson et al (2009) to study off-axis underground soil pressure from surface impact loads, and results were found to be in good agreement with mathematical solutions provided by Laturelle (1990), Guan and Novak (1994) and Boussinesq's equation (ALA, 2001). Pichler et al. (2005) developed a three dimensional (3D) finite element quarter model for a gravel buried steel pipeline subjected to a surface impact, and the model was assessed by using real-scale impact experiments. A 3D finite element model of the soil, steel pipelines, and concrete structures was developed by Zarghamee et al. (2009) using ABAQUS to evaluate various load drop scenarios for a nuclear power plant life extension project. ABAQUS was used by Yang and Komvopoulos (2005) to study a dynamic contact problem of a rigid sphere impacting the surface of an elastic homogeneous half-space.

COMPARISON OF METHODS

Comparisons of the various methods used are complicated somewhat by the fundamental differences in their approaches. The scaled energy approach uses field data from pile driving activity, deep soil compaction, etc., from which the peak particle velocity or stress at distance from the point of impact can be determined directly. There is no intermediate step for evaluation of the impulse that allows comparison, say, with the impulsively loaded elastic half space FE or analytical response.

When the impulse function has a reliable definition, closed form analytical solutions and finite element analyses apply. Closed form results from the literature for surface and subsurface elastic half space response are usually based on integral transform methods and are provided as charts or graphs for limited discrete or continuous ranges of non-dimensional system parameters (Guan and Novak, 1994; Laturelle, 1990, 1991; Georgiadis et al, 1999). These references contain details for the development of the integral transforms so that their case-by-case inversion for particular system parameters can, although difficult, be performed in principle. Their solutions show favorable comparisons with FE results; when implemented correctly, either the FE or integral transform result will provide reliable near-surface and subsurface free-field stresses. Considering, however, the effort as well as the mathematical understanding required to successfully invert the integral transforms, the FE method is, in general preferred, except when system parameters enable use of the charted or graphed response results from the literature. Both the FE and closed form methods apply when a good approximation of the surface impulse is available.

Surface impulses have been approximated by any one of several methods: conservation of momentum ($M\Delta V = F\Delta T$), energy balance based on empirically determined projectile penetration, and energy balance based on soil ultimate bearing resistance. Empirically determined penetration distances are unreliable design input for calculation of impulse and subsequent free-field stress determination, because the formulae are only applicable to penetrations in the range of 3 or more calibers (Young, 1997). Missiles which satisfy this criterion will have relatively small footprints and will transmit their energies to the soil by local penetration, with minimal soil wave transmission away from the penetration. In such cases, the design approach is simply to assure that the penetration depth is less than the burial depth to the pipe crown (plus a small additional distance to serve as a factor or safety) so that the pipe is not directly impacted by the missile. On the other hand, a missile with large footprint can penetrate only a fraction of its footprint dimension into the soil, thereby invalidating the formulae used to calculate the penetration distance. Thus, impulse load will be difficult to predict for missiles with either small or large footprint and such calculated impulses should be applied with caution. The ultimate dynamic bearing pressure approach is problematic in that the soil dynamic ultimate bearing resistance is a function of the confinement provided by the surrounding soil half-space. Based on these observations, scaled energy data, which are independent of any explicit impulse function determination, are considered the better free-field response estimation tool when impulse definitions need to be calculated by any of the above methods. In the use of these scaled energy data, care must be taken to use data based on parameters consistent with the application at hand. Deep soil compaction scaled energy data, for example, would be more applicable to drop of a large electric motor, say, than would scaled energy data from pile driving activity.

It is known from the literature (Dowding, 1996) that soil response to impact loading is characterized by responses consisting of a mixture of wave types (Rayleigh, compression, shear, etc.) in the near vicinity of the impact loading. These various wave types attenuate at distance, with the Rayleigh wave attenuating at a lower rate, so that, at distance, the underground, near surface response is dominated by the Rayleigh wave. Rayleigh wave response attenuates in accordance with the relation

$$\sigma_2 = \sigma_1(R_1/R_2)^{1/2}e^{-\alpha(R2-R1)} \tag{9}$$

where σ_1 and σ_2 are stresses at distances R_1 and R_2 respectively. The form of this equation is precisely consistent with the FE analysis results of Johnson et al (2009), which show, beyond $R/R_0 = 5$, strong linear behavior on the log-log plot for stress vs. distance. For R/R_0 in the range below 5, the response departs from the prediction of Eq. 9. This departure is attributed to St. Venant effects, which were demonstrated by Awrejcewicz and Pyryev (2003 to become significant for the Rayleigh surface wave response when R/R_0 is less than 3. Also, in this range, the horizontal restraints on the axisymmetric boundary of the FE model used by Johnson et al (2009) produce local spurious effects which influence the response in the vicinity of the impact footprint. In conclusion, properly performed FE analyses provide reliable free-field response results at sufficient distance from the impact loading, and are preferred over scaled energy data plots representative of Eq. 9 when a result dependent upon and reflecting impact duration and interface footprint geometry is desired.

CASE STUDY OF HAND METHODS

Consider two practical examples with the soil density $\rho = 120$ pcf, shear wave velocity $V_s = 1000$ fps. Dimensions in Figure 2 are set as z = 6 ft, b = 0 ft (on-axis drop) or b = 25 ft (off-axis drop). Other parameters are as defined in Table 2.

Results of maximum surface impulse F_{max} are demonstrated in Table 3. In Case 1, the maximum impulse force calculated using conservation of momentum concept (883 kip) is about 50~100% higher than the ones obtained using the conservation of energy (424 kip & 601 kip). On the other hand, impulse duration calculated based on momentum (0.051s) is about 50~75% of the ones based on energy (0.105s & 0.074s). In Case 2, the energy method leads to significant impulse loads up to 3796 kip while the momentum method gives only 765 kip. It should be noticed that in Case 2, the penetration depth is only 1/30~1/10 of the impact footprint, which indicates the energy method may not be applicable.

Table 2: Missile parameters

Case ID	Drop weight W (kip)	Drop height h (ft)	Drop footprint L (ft)
Case 1	10	80	1 × 1
Case 2	5	40	3 × 3

Table 3: Surface Impulse F_{max}

Method to determine impulse load	Case 1			Case 2		
	F_{max} (kip)	Duration (s)	Penetration depth (ft)	F_{max} (kip)	Duration (s)	Penetration depth (ft)
Conservation of Momentum, Eq (1)	883	0.051	NA	765	0.021	NA
Conservation of Energy, Eq (2) & Eq (3)	424	0.105	3.771	3796	0.0042	0.105
Conservation of Energy, Eq (2) & Eq (4)	601	0.0742	2.66	1260	0.0125	0.317

Table 4: Vertical pressure q_{max} at the top of pipeline

Case ID	Maximum impulse (kip)	On-axis drop (psi)				Off-axis drop (psi)	
		Eq (5)	Eq (6)	Eq (7)	Eq(8)	Eq (5)	Eq (6)
Case 1	883	81	648	78	127	1.34	1.93
Case 2	765	70	180	50	66	1.16	0.563

Given maximum impulse forces, F_{max} ,determined by the conservation of momentum as 883 kip and 765 kip respectively for Case 1 and Case 2, on-axis and off-axis vertical pressures q_{max} on the top of the pipeline are calculated using various Boussinesq or empirical equations and presented in Table 4 for comparison. Results from the various approaches are rather consistent and within the same order of magnitude, with the exception of Eq (6) (Gupta and Saigal 2003) applied in an on-axis Case 1 drop. It is also noted that the application of pressures calculated from on-axis locations to off-axis locations is conservative.

PIPE-SOIL INTERACTION EFFECTS

During an event of surface impact, the presence of buried pipes inside the elastic half-space causes disturbance of the free-field stress profile. Therefore, the problem of evaluation of buried pipes for surface impact loads can be considered in two parts: one involving the determination of the pressure at the top of pipe for statically applied surface loads accounting for the pipe-soil interaction effects, and the other involving consideration of amplification effects due to dynamic nature of load.

Poulos and Davis (1991) provides analytical solutions for the stress distribution around lined cavities in elastic half-space subjected to surface pressure for slip and no-slip conditions at the cavity-soil interface. These solutions reflect the change in stress distribution due to the difference in the stiffness of the cavity and the surrounding soil. The radial stress distribution around the cavity is non-uniform and depending on the relative stiffness of the cavity and the soil, the radial stress at the crown of the pipe (i.e., vertical stress typically used for design evaluations of buried pipes) may be amplified or reduced when compared to the free-field static stresses.

Gupta and Saigal (2003) applied the concept of stress distribution around buried pipes for uniform static surface loads (Poulos and Davis, 1991) to surface impact scenarios and validated the modifications by comparison with results from finite element method and with observations from experiments. For static loads applied at the surface over a finite area, the free-field stress at any point in the elastic half-space can be calculated using Eq. 6. Gupta and Saigal (2003) proposed that the stress distribution around buried pipes subjected to loads applied at surface over finite areas can be reasonably estimated by replacing the uniform static surface pressure in the analytical solutions given in Poulos and Davis (1991) with the free-field stress computed using Eq. 6.

For buried pipes in typical nuclear power plant applications, the relative stiffness of the buried pipe and surrounding soil is such that the radial stress at the crown of buried pipe is reduced compared to the stress in the free-field. Therefore, in most cases, discounting the pipe-soil interaction effects would lead to conservative results. However, in some cases, the relative stiffness of the pipe and surrounding soil may amplify the stresses around the pipe by as much as 40% compared to the free-field stresses and this aspect must be appropriately considered in the design evaluations (Poulos and Davis, 1991). Since the dynamic amplification effects are insignificant for most cases, using a dynamic load factor greater than 1.0 to account may indirectly account for such situations. However, in other cases this may lead to over-conservatism and provide unnecessary constraints to the load lifts and construction lift paths during nuclear power plant modifications.

CONCLUSIONS AND RECOMMENDATIONS

In this paper, a comprehensive literature search and evaluation is carried out for a buried pipes subjected to a surface impact load. Observations and recommendations are summarized below:

1. Prior to a detailed closed form analytical solution (integral transform methodologies) or finite element solution, approximate simpler methods should always be implemented.
2. For large footprint impacts, avoid conservation of energy approaches using calculated penetration from empirical formulas.
3. Application of pressures calculated for on-axis drop, to off-axis locations within the soil may be over conservative.
4. Where scaled energy data is used, select data from phenomena as similar as possible to the impact under consideration (footprint dimension, soil characteristics, energy range, etc.).
5. Properly preformed FE analyses provide reliable free-field response at sufficient distance for the impact loading, and are preferred over the scaled energy method.
6. A dynamic load factor between 1.0~1.2 is generally sufficient to ensure conservatism in practice.
7. For buried pipes in typical nuclear power plant applications, discounting the pipe-soil interaction effects will in most cases lead to conservative results.

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