Predicting Performance of Buried Metal Conduits

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ABSTRACT

The performance of buried metal conduits described in this paper was predicted by using the finite-element program Culvert Analysis and Design (CANDE) as modified by research done at Purdue University. The essential aspects involved in predicting conduit behavior are revealed through discussion of the following topics: (a) effects of using different soil models in the analysis, (b) effects of different soil-conduit interface conditions, (c) importance of the sequence in placement of soil layers, (d) yielding and buckling of the conduit wall, and (e) some applications of the analysis to practice. Buckling of buried metal conduits has been shown to be an important failure mode. In the absence of buckling, yielding in the conduit walls can result in a favorable redistribution of soil pressures, thereby permitting the conduit to support the overburden loads more efficiently. A useful procedure to track the potentially beneficial effects of wall yielding is presented. Diagrams are given for estimating reliably the maximum thrust and the maximum elongation in vertical diameter of buried circular and elliptical conduits.

Some fundamental response characteristics of buried metal conduits that have evolved from research done at Purdue University (1-5) are summarized. For the sake of concentrating on the elucidation of fundamental behavior patterns under overburden loads, the sample problems given in this paper are limited to circular and elliptical conduits. These sample problems were solved by using the CANDE code (6) as modified by research done at Purdue University. The importance of these latter modifications and a comparative study of CANDE and other computer codes are well documented elsewhere (3,7).

The essential aspects involved in predicting conduit behavior are revealed through discussion of the following topics: (a) effects of using different soil models in the analysis, (b) effects of different soil-conduit interface conditions, (c) importance of the sequence in placement of soil layers, (d) yielding and buckling of the conduit wall, and (e) some applications of the analysis to practice. The effects of live loads and of loads applied during compaction are not considered in this paper.

PROBLEMS STUDIED

The first group of problems was selected to study the effect on the conduit response of using different soil models. In this group of problems, a 10-ft-diameter, 18-gauge 2 2/3 x 1/2 corrugated steel conduit (see Table 1 for sectional properties) with 25 ft of soil cover above the crown (i.e., 30 ft above the springline) was analyzed. The soil models used include linear-elastic, overburden-dependent, extended Hardin, and Duncan-Chang models. The parameters used to represent these soil models are listed in Table 2.

The second group of problems was selected to study the effects of different soil-conduit interface conditions on the conduit response. Two 10-ft-diameter circular steel conduits, 18-gauge 2 2/3 x 1/2 and 12-gauge 6 x 2 corrugated steel pipe, and two states of compaction, a moderately loose sand and a moderately dense sand, were analyzed. Comparisons of conduit responses were made between two different interface conditions, fully bonded versus slip with a limiting friction coefficient of 0.5. The soil parameters used to define the Duncan-Chang soil model for the two sands are given in Table 3.

The third group of problems was selected to demonstrate the importance of the sequence in placement of soil layers. Both circular and elliptical

TABLE 1 Sectional Properties for Conduits Studied

	Sectional Property		
Conduit	Area ($in.^2/in.$)	Moment of Inertia ($in.^4/in.$)	Section Modulus ($in.^3/in.$)
1-gauge 6 x 2 in.	0.3432	0.1658	0.1458
7-gauge 6 x 2 in.	0.2282	0.1080	0.0989
12-gauge 6 x 2 in.	0.1296	0.0604	0.0574
18-gauge 2 2/3 x 1/2 in.	0.0516	0.0015	0.0055

TABLE 2 Parameters for Various Soil Models Used in Group 1 Problems

	Model			
Parameter	Linear Elastic	Overburden Dependent	Extended Hardin[a]	Duncan-Chang[a]
E (psi)	700	_[b]	n.a.	n.a.
μ	0.33, 0.45	0.32, 0.45	n.a.	n.a.
μ_{min}	n.a.	n.a.	0.20	n.a.
μ_{max}	n.a.	n.a.	0.495	n.a.
q	n.a.	n.a.	3.75	n.a.
S_1	n.a.	n.a.	1,038	n.a.
C_1	n.a.	n.a.	814,000	n.a.
a	n.a.	n.a.	-1.75	n.a.
ϕ (degrees)	n.a.	n.a.	n.a.	35
K	n.a.	n.a.	n.a.	920
n	n.a.	n.a.	n.a.	0.79
R_f	n.a.	n.a.	n.a.	0.96
G	n.a.	n.a.	n.a.	0.37
F	n.a.	n.a.	n.a.	0.12
D	n.a.	n.a.	n.a.	10.5

Note: n.a. = not applicable.

[a]Based on Lade's triaxial compression test data (8) for moderately dense Monterey No. 0 sand.

[b]The overburden pressures (psi) and their corresponding E-values are as follows: 5, 550; 10, 750; 15, 850; 20, 1000; 25, 1100; 30, 1150; 40, 1300; 50, 1400.

TABLE 3 Duncan-Chang Parameters for Group 2, 3, and 4 Problems

Parameter[a]	Moderately Loose Sand[b]	Moderately Dense Sand[c]	Dense Sand[d]
ϕ	30	35	42
K	280	920	600
n	0.65	0.79	0.4
R_f	0.93	0.96	0.7
G	0.35	0.37	0.35
F	0.07	0.12	0.2
D	3.50	10.5	15

[a]The stress-strain relation of granular soils at the same relative compaction is dependent on particle size, shape, angularity, surface roughness, and size distribution. Therefore, soil parameters for the Duncan-Chang model must be used as a set. For a listing of representative values, see report by Wong and Duncan *(9)* or Table 2.2 of report by Leonards et al. *(3)*.
[b]Relative compaction (RC) compared with optimum density in the standard AASHTO (T-99) test = 92 percent.
[c]RC = 97 percent.
[d]RC = 100 percent.

1-gauge 6 x 2 steel conduits at a span of 25 ft were analyzed. The soil used was a dense sand the parameters of which are given in Table 3. Comparisons were made between two loading sequences--favorable and unfavorable from the standpoint of inducing maximum bending moments in conduits with shallow cover. The layer sequences were chosen not to simulate actual construction practices but to provide a range that would bracket a majority of such procedures. Examples of the sequences adopted for this purpose are shown in Figure 1 for a circular conduit and in Figure 2 for an elliptical conduit with rise/span ratio (R/S) of 0.25 (rise is defined as the vertical distance from the springline to the crown).

An extensive review of the performance of buried conduits with special emphasis on buckling failure modes (2) provided the basis for the discussions of yielding, buckling, and collapse loads. The fourth group of problems was selected to demonstrate the consequences of yielding in the conduit wall. Two circular corrugated steel pipes, 1-gauge 6 x 2 at a

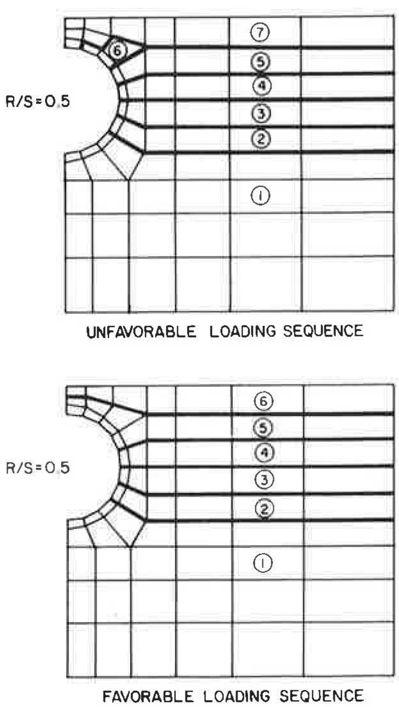

FIGURE 1 Favorable and unfavorable layer sequences for circular conduits.

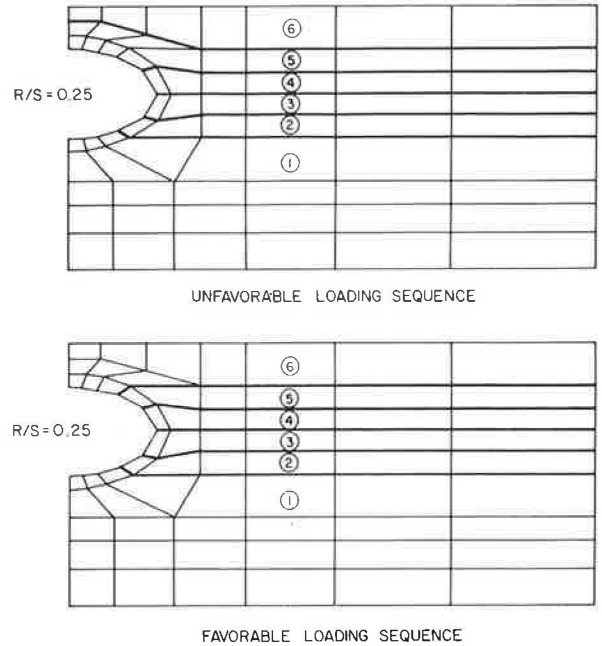

FIGURE 2 Favorable and unfavorable layer sequences for an elliptical conduit with R/S = 0.25.

span of 25 ft and 18-gauge 2 2/3 x 1/2 at a span of 10 ft, were analyzed. The soil used was a moderately loose sand the parameters of which are given in Table 3. In addition, experimental data on full-size culverts were utilized to examine the relation between yielding, buckling, and collapse loads.

The fifth group of problems was analyzed to provide a basis for comparing some provisions of a proposed Code of Practice for designing buried steel structures (4). The study was restricted to structures backfilled with granular soils because (a) large-span steel conduits were of primary interest and these are almost always constructed with select backfill soils and (b) the empirical data base to characterize the stress-strain behavior of granular soils is more extensive and much more reliable than that of other types of soils. The influence of the following factors on the maximum thrust and conduit deflection was evaluated:

1. Conduit shape: circular and elliptical,
2. Conduit wall stiffness: covering the range of available corrugated steel plates,
3. Conduit span: 10 to 35 ft,
4. Height of cover: 3 to 75 ft,
5. Level of soil compaction: fair to good,
6. Sequence in placement of soil layers: favorable and unfavorable, and
7. Interface conditions and soil models.

RESULTS AND DISCUSSION

Effects of Using Different Soil Models

It has often been suggested that there are no significant differences in the predicted conduit response if different soil models are used, provided that the "correct" magnitudes of the relevant soil parameters are adopted; in fact, isotropic, linear-elastic soil models have been considered satisfactory for predicting the performance of buried conduits (10-13), although their shortcomings are gradually being recognized (3,14).

TABLE 4 Results from Finite-Element Solutions of Group 1 Problems

	Model					
	Linear Elastic		Overburden Dependent			
Response	$\mu = 0.33$	$\mu = 0.45$	$\mu = 0.32$	$\mu = 0.45$	Extended Hardin	Duncan-Chang
P_{max} (kips/ft)	17.9	16.7	19.6	16.4	13.4	14.7
M_{max} (ft-lb/ft)	71	21	27	13	8	27
ΔY (%)[a] at H = 25 ft	-2.78	-0.55	-1.08	-0.07		+0.18
$\epsilon_{max}/\epsilon_y$[b]	1.37	0.92	1.11	0.84	0.67	0.80

[a]ΔY (%) is percent change in vertical diameter; positive values indicate elongations.
[b]ϵ_{max} = maximum strain in conduit wall, ϵ_y = yield strain of steel (= σ_y/E = 0.0011).

Some results from the study of the first group of problems (i.e., differences between soil models) are summarized in Table 4. As may be seen from Table 4, the differences in the maximum bending moment and the changes in vertical diameter are significant in all cases, whereas the differences in the maximum thrust are more modest.

The percent change in vertical diameter as a function of fill height is shown in Figure 3. The use of a linear-elastic soil model with Poisson's ratio $\mu = 0.33$ does not produce elongation of the vertical diameter during construction, which is an unrealistic result; on the other hand, with $\mu = 0.45$, the linear-elastic model gives almost identical peaking effects during construction as does the Duncan-Chang model. However, the rate at which the diameter shortens after the fill height is above the crown is much more rapid for the linear-elastic model than is the case for the Duncan-Chang model. This demonstrates that a linear-elastic soil model that is equivalent for one aspect of the problem is not equivalent for another aspect. Similar conclusions may be drawn from Table 4 with respect to the overburden-dependent model. With $\mu = 0.32$, the calculated thrust is too large but the bending moment is realistic; with $\mu = 0.45$, the calculated thrust is reasonable but the bending moment is unrealistically low.

The difference between the extended Hardin and Duncan-Chang soil models may be seen from curves 5 and 6 in Figure 3. The soil parameters for these two models were obtained from the same triaxial test data reported by Lade (8) by using procedures recommended in other studies (9,15-17); the observed differences reflect inherent differences in the models and not the errors associated with correlations between the soil parameters and the results of classification tests. Although the trends are similar, it is believed that the responses indicated by the extended Hardin model are too small.

Effects of Different Soil-Conduit Interface Conditions

The results from the study of the second group of problems are as follows. Figure 4 shows the effects of interface conditions on the conduit response (in terms of change in vertical diameter versus fill

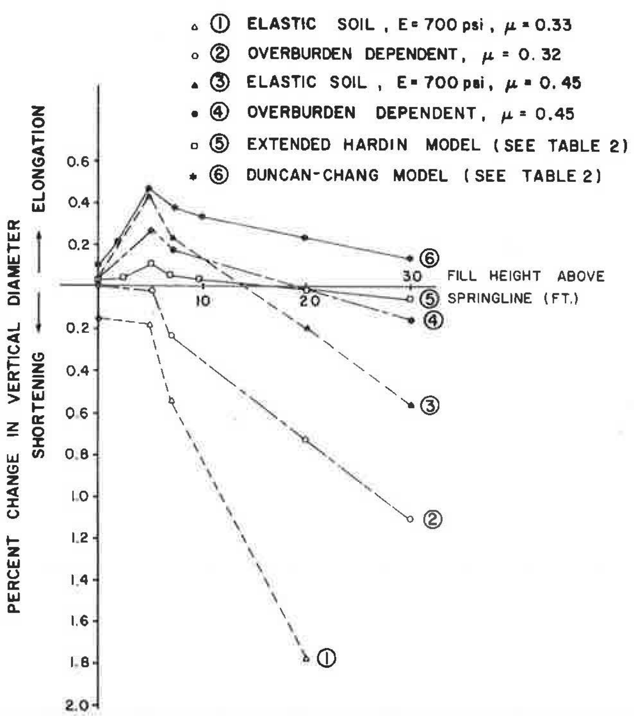

FIGURE 3 Effect of soil model on change in vertical diameter versus fill height for 10-ft-diameter steel conduit.

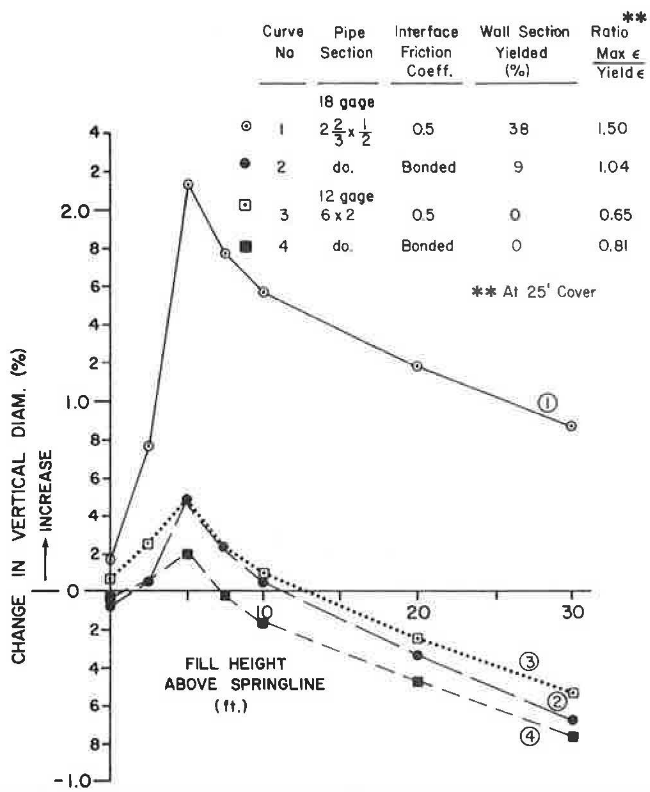

FIGURE 4 Effects of interface conditions on conduit deformation: moderately loose sand.

height) for the two conduits backfilled with a moderately loose sand. As can be seen from Figure 4, the 18-gauge 2 2/3 x 1/2 conduit increases in diameter more than 4 times as much if a limiting friction coefficient of 0.5 is used than if the interface is fully bonded; with 25 ft of cover the vertical diameter was still extended by 1 percent when slip was allowed, whereas for the fully bonded condition the diameter had decreased by 0.7 percent. A similar trend was obtained for both conduits backfilled with a moderately dense sand, which is shown in Figure 5. It may be noted from Figures 4 and 5 that although the effects of allowing interface slip to occur are more pronounced in the case of the more flexible pipe, they are still significant for the stiffer conduit.

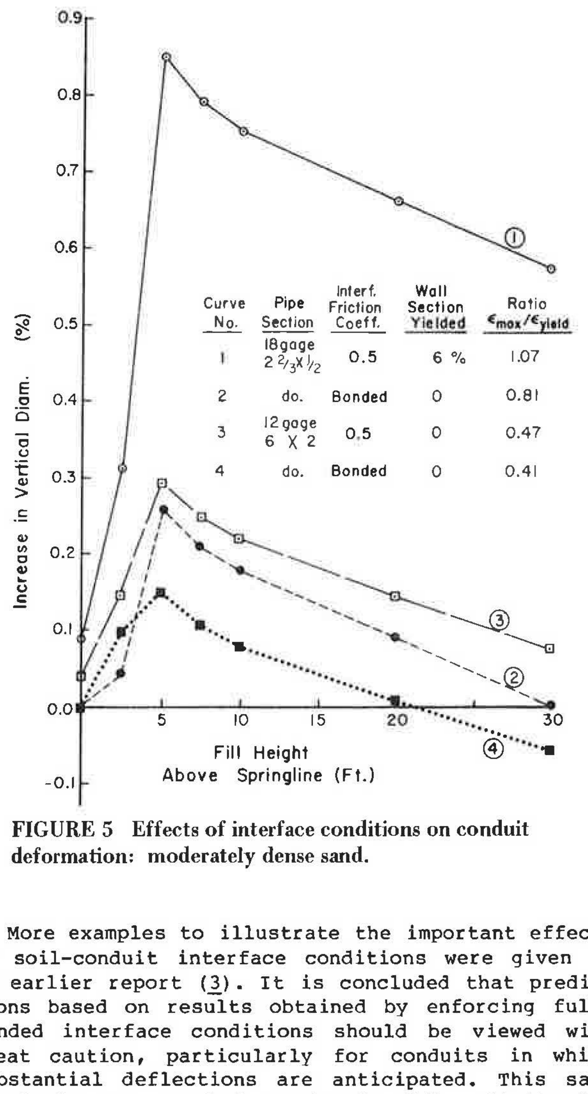

FIGURE 5 Effects of interface conditions on conduit deformation: moderately dense sand.

More examples to illustrate the important effects of soil-conduit interface conditions were given in an earlier report (3). It is concluded that predictions based on results obtained by enforcing fully bonded interface conditions should be viewed with great caution, particularly for conduits in which substantial deflections are anticipated. This same conclusion was reached previously (13), albeit with the use of a linear-elastic soil model.

Importance of Sequence in Placement of Soil Layers

Experience has brought into focus the sensitivity of conduit response to modest variations in the sequence of placing soil in layers around and over the conduit. It has been stated that, up to now, engineering computations have provided little assistance in evaluating the effects of compacting soil up to the level of the crown (18), yet it appears evident that when predictions are compared with field measurements, failure to model the sequence of soil placement closely could invalidate the conclusions that are drawn. Some effects of placement sequence were analyzed in the third group of problems. Figure 6 shows that the maximum bending moments are strongly influenced by the sequence of soil placement. Even for the case of a dense sand backfill, the difference in bending moment is 75 percent when R/S = 0.25 and 300 percent when R/S = 0.50 (circular conduit) for the two placement sequences shown in Figures 1 and 2. Figure 7 shows the effects of placement sequence on the elongation in vertical diameter; it is seen that these effects are indeed significant. The data in Figure 7 were replotted in Figure 8 assuming as a datum the point at which soil backfill reached the crown. Figure 8 gives a badly distorted picture of the system response. Similar phenomena are observed from the standpoint of inducing maximum thrusts except that a layer sequence that is unfavorable for bending moment is usually favorable for thrust, and vice versa. These results clearly show the necessity of modeling the soil placement sequence as closely as possible to obtain meaningful comparisons between predicted and observed performance of buried conduits.

Yielding, Buckling, and Collapse Load

Because of the initially unsupported condition of the conduit and the nonuniform loading imposed during placement of soil layers, local yielding may develop in the conduit wall during construction. Such local yielding is not necessarily detrimental, provided that the conduit shape is not too asymmetrical and the soil backfill has good support characteristics (well-compacted granular soil). In fact,

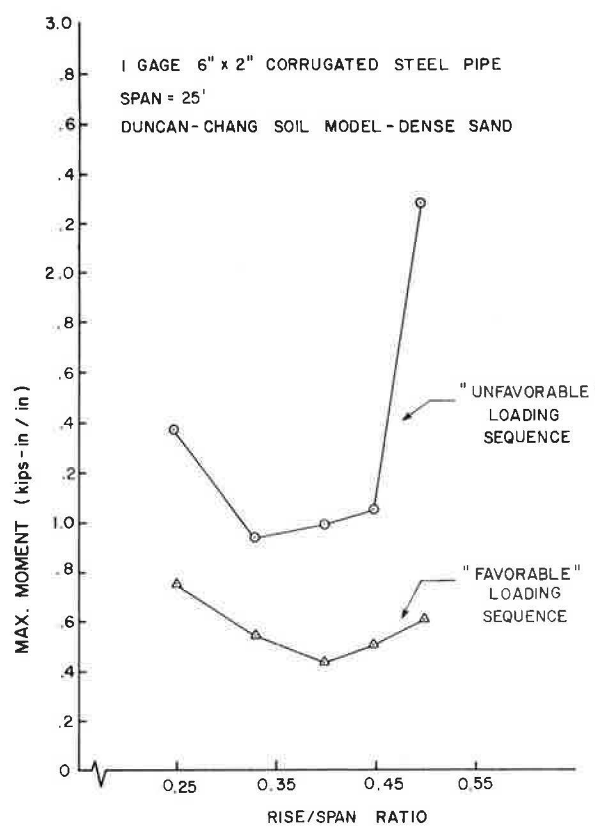

FIGURE 6 Effect of soil layer sequence on relation between bending moment and rise/span ratio.

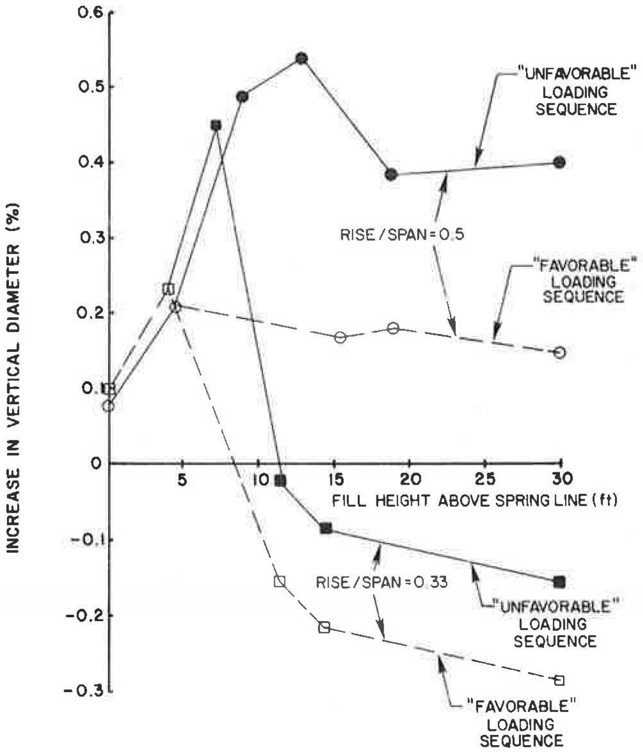

FIGURE 7 Effect of soil layer sequence on conduit deformation.

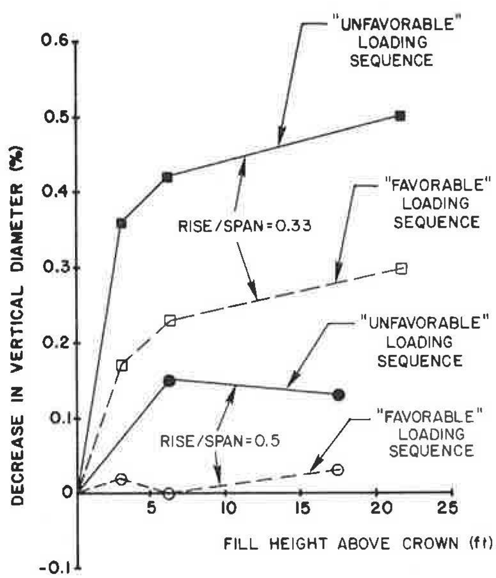

FIGURE 8 Effect of soil layer sequence on conduit deformation (datum at level of crown).

local yielding during construction may prestress the conduit favorably with respect to the subsequent reversal in bending moment at the crown, thereby contributing to more efficient load-support characteristics as the height of cover is increased. In any case, from the standpoint of predicting culvert response, it is essential that the effects of local yielding be accounted for properly.

When nonlinear soil behavior is being accommodated (e.g., the Duncan-Chang soil model), the results are sensitive to the manner in which yielding in the conduit wall is modeled. It has been found necessary to satisfy equilibrium, compatibility, and the stress-strain relations closely, and this can only be accomplished if iteration is continued to convergence (3). The modified CANDE code is the only well-documented computer code known to the authors that satisfies all these requirements.

A useful way to track the potential consequences of yielding in the wall section is to examine how closely the amount of yielding approaches the condition of a fully plastic hinge. A plastic hinge will form if the following criterion is satisfied [for a rectangular section subjected to the combined action of thrust (P) and moment (M)]:

$$(M/M_p) + (P/P_p)^2 = 1 \qquad (1)$$

where M_p is the fully plastic moment of the section, the limiting moment when the section is subjected to pure bending, and P_p is the squash load of the section in the absence of bending moment. In this equation, the value of M is the moment about the centroidal axis of the wall section; hence the results from codes that calculate M about other axes are inappropriate for use in this equation.

Valuable insight into the response of a conduit to increasing heights of cover can be gained by plotting M/M_p versus P/P_p, as defined in Equation 1. An example of such a plot is shown in Figure 9 (for clarity only one point in each wall section has been plotted; in practice, several key points can be followed in the same diagram). The safety factor against formation of a plastic hinge (F_p) is given by the ratio of the distances $\overline{AO}/\overline{OB}$ in Figure 9. It can be shown that this ratio is identical to the value of F_p defined by Duncan (19) as

$$F_p = 0.5(P_p/P) \times \left\{[(M/M_p)^2 \times (P_p/P)^2 + 4]^{1/2} - [(M/M_p) \times (P_p/P)]\right\} \qquad (2)$$

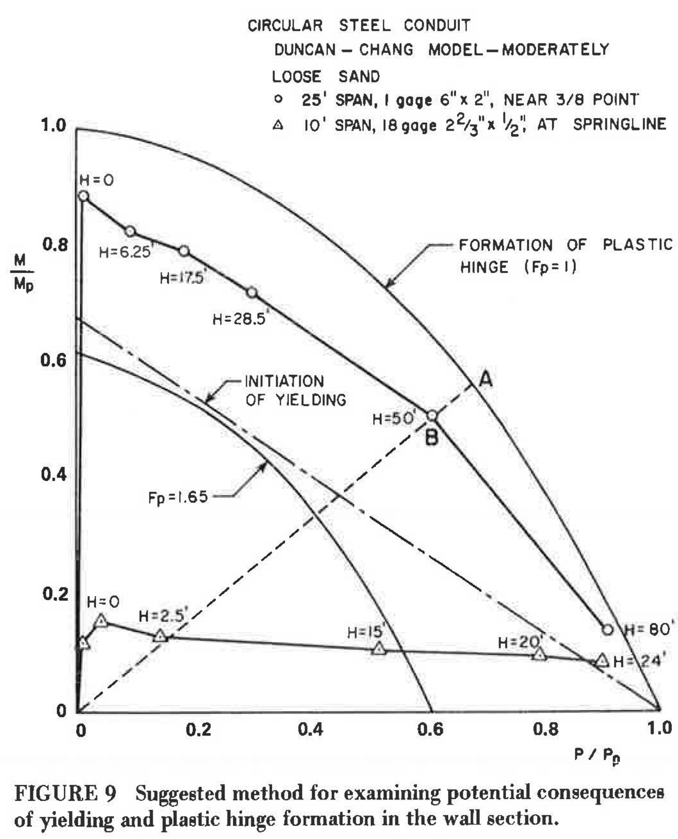

FIGURE 9 Suggested method for examining potential consequences of yielding and plastic hinge formation in the wall section.

Considering first the results shown in Figure 9 from the conduit with a 25-ft span, the lowest safety factor against formation of a plastic hinge, $F_p = 1.1$, occurs during construction when the fill height is near the crown (H = 0); at this time, a substantial fraction of the wall section has yielded. As the height of cover is increased, the thrust increases, but the redistribution of soil pressure causes a corresponding decrease in bending moment such that F_p actually increases (to a value of 1.2 at H = 28.5 ft). Further increase in fill height causes F_p to decrease until at H = 50 ft it is again reduced to 1.1. If there is no danger of buckling, the fill height could be increased at least to 80 ft before the squash load in the wall section is approached. Thus, the potential for buckling is a key design consideration for large-span conduits.

In the case of the 10-ft-diameter culvert, bending is not a significant factor provided the backfill is granular and reasonably well compacted. Increases in fill height manifest themselves largely as increases in thrust; thus, it is not the height of soil cover but the span of the conduit that plays the key role in controlling the mode of soil-conduit interaction.

Also shown in Figure 9 is a line defining the boundary for the initiation of yielding in the wall section and a curve showing $F_p = 1.65$, which has been recommended for design purposes (19). It is seen that $F_p = 1.65$ inhibits even the slightest initiation of yielding during initial backfilling, a requirement that may be too restrictive in all cases.

Available computer codes that can accommodate nonlinear soil behavior, slip at the soil-conduit interface, and yielding of the conduit wall section are based on small strain theory and hence cannot account for buckling behavior. Thus, in the past, attention has been directed toward examining other potential failure modes: (a) wall crushing, (b) seam separation, (c) yielding of the wall section, and (d) excessive deflections. An extensive review of buckling failures in buried conduits (2) revealed that buckling is an important failure mode, that it can occur at stress levels below yielding or after yielding has been initiated, that it can occur at deflections less than 5 percent of the span, and that current methods of analysis are utterly incapable of predicting the buckling load, that is, whether buckling will result in local crimping of the conduit wall or in a potentially catastrophic snap-through buckle at the crown or, if local buckling occurs, how close the conduit is to collapse. Some experimental results supporting these findings are shown in Figures 10 and 11, from which it may be seen that wall crimping due to local buckling (like seam slippage) may actually contribute to an increase in the collapse load. It may well be that the main benefit of stiffening ribs is to guard against snap-through buckling at the crown rather than to provide additional resistance to bending.

Inability to estimate the collapse load of buried conduits accounts for by far the largest uncertainty in the use of analytical methods to design buried conduits, especially if concepts of ultimate limit state and probability of failure are to be incorporated in the design. More research to define and predict the collapse load is urgently needed.

Application of Analysis to Practice

The fifth group of problems was analyzed to compare the results with the provisions of a proposed new design code for buried steel structures, which was part of a study conducted for the Ontario Ministry of Transportation and Communications (4).

In an earlier study (3), some results of finite-element analyses to obtain the maximum thrust in the conduit wall were compared with those calculated by using ring-compression theory (22) and the soil-conduit interaction (SCI) procedure (19), as shown in Figure 12. In general, ring-compression theory tends to underestimate the maximum thrust, whereas the SCI procedure tends to overestimate it. On the other hand, use of the weight of soil between vertical planes that bound the crown and the springline (Figure 13) appears to give a reasonable approximation to the calculated maximum thrust. This weight of overburden (W) may be expressed as

$$W = \gamma D_H (0.1R + 0.5H) \tag{3}$$

where

γ = unit weight of soil,
H = height of soil cover,
D_H = span, and
R = rise (the vertical distance from the springline to the crown).

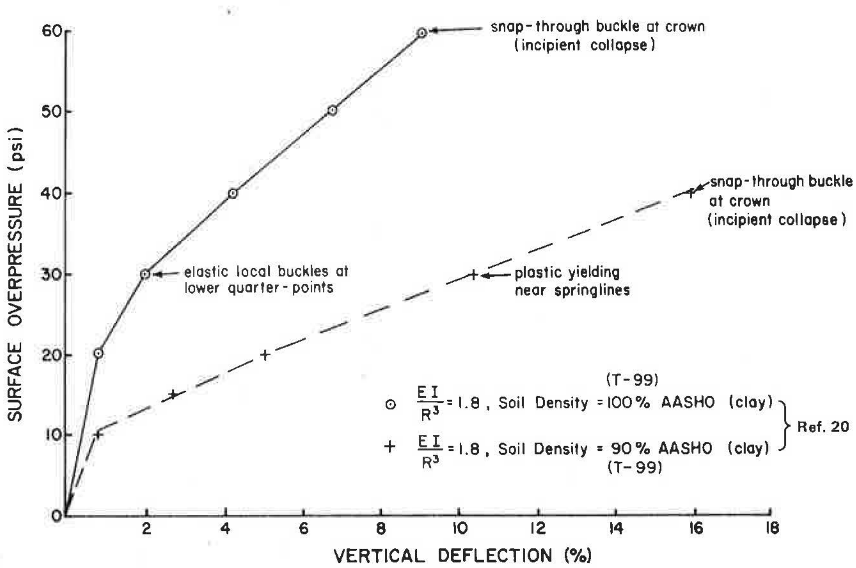

FIGURE 10 Load-deflection behavior of full-scale conduits: data from Howard *(20)*.

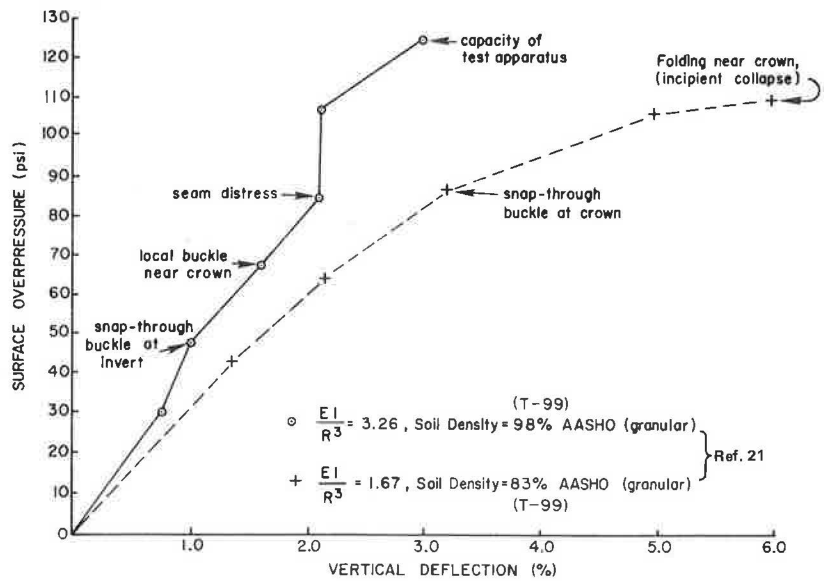

FIGURE 11 Load-deflection behavior of full-scale conduits: data from Watkins and Moser *(21)*.

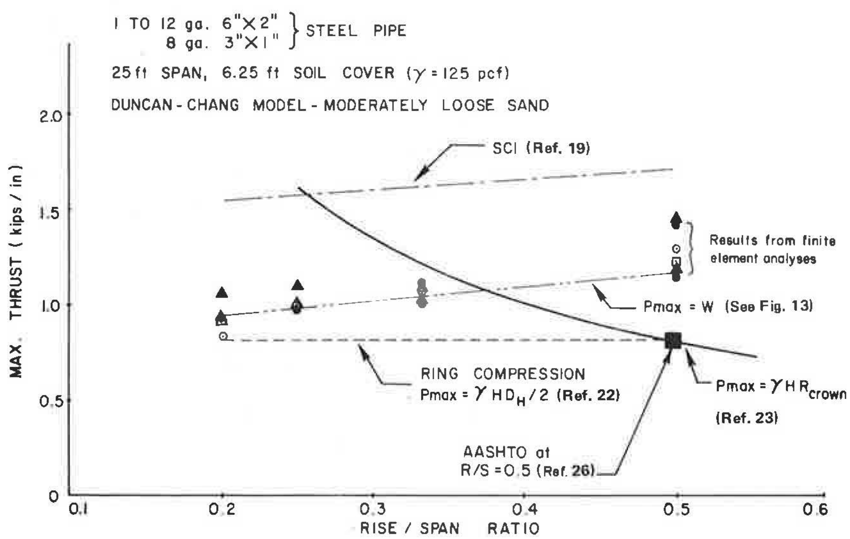

FIGURE 12 Effect of rise/span ratio on maximum thrust.

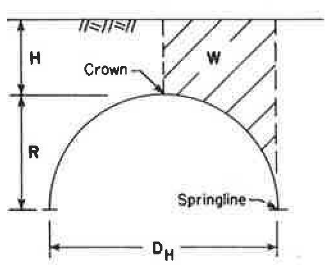

FIGURE 13 Weight of overburden contained between vertical planes rising upward from crown and springline.

If the radius of curvature at the crown is utilized instead of the half-span, the trend in the curve of maximum thrust versus rise/span ratio is opposite to that obtained by using finite-element analysis. The precision with which P_{max} can be estimated by using the weight of soil above the springline (Figure 13) can be improved by back-calculating an arching factor depending on the height of soil cover above the crown.

The preliminary Ontario code (1981) provided that the maximum thrust acting on the conduit walls due to overburden loading (i.e., dead load from backfill material) should be calculated as follows:

$$T_D = \mu_1 \cdot W \tag{4}$$

where

T_D = maximum thrust,
W = weight of overburden contained between vertical planes rising upward from the crown and the springline (Figure 13), and
μ_1 = arching factor.

A comparison of arching factors proposed in the preliminary code with those calculated from the problems in the fifth group is given in Figure 14. To prepare this figure, arching factors were back-calculated from Equation 4; the maximum thrusts were

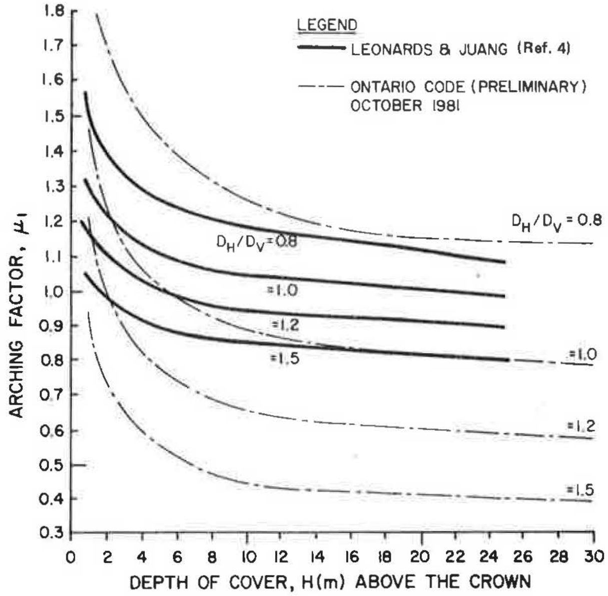

FIGURE 14 Arching factor versus depth of soil cover for circular or elliptical conduits.

obtained from finite-element analyses of the buried conduits over a range in diameter up to 35 ft, a range in the ratio of horizontal to vertical span (D_H/D_V) of 0.8 to 1.5 (i.e., R/S ranges from 0.33 to 0.63; $D_H/D_V = 1$ or R/S = 0.5 implies a circular pipe), and depths of soil cover above the crown up to 75 ft. It is noted that the values of μ_1 given in the preliminary code were generally too low, especially for the flatter ellipses. Although the thrusts calculated by using values of μ_1 from Figure 14 agree well with those measured in the field [e.g., by Katona et al. (6) and Spannagel et al. (24)], the approved code (25, pp. 277-288) departed from the use of Equation 4 to calculate T_D in order to accommodate nonelliptical conduit shapes and to utilize load and material factors derived from past experience.

The arching factor μ_1 depends on the conduit span and stiffness, the type and state of compaction of the backfill, and the height of cover. However, because of the general relation between conduit span and stiffness, if μ_1 is plotted versus height of cover, the only other factor affecting the results is the stiffness of the backfill. Figure 14 was drawn for moderately compact sand; hence in this respect the values of μ_1 will be slightly conservative if better compaction is achieved in the field.

Figure 14 shows a general reduction in values of μ_1 with increasing height of cover and decreasing rise/span ratio, although once the cover reaches a height of about 30 ft, μ_1 becomes sensibly constant. For common shapes and heights of cover, values of μ_1 are usually greater than 1, especially for very shallow cover. Neither the AASHTO (26) nor the American Iron and Steel Institute (AISI) (27) recommendations are in keeping with these trends.

The Ontario code also has a criterion for controlling deformations during backfilling. It has been pointed out that the change in shape of the conduit during backfilling has an important influence on the potential for buckling as well as on the ultimate load capacity (2). Various approaches have been taken to maintain the change in shape within tolerable limits. For example, the AISI Handbook (27) states that "construction procedures must be such that severe deformations do not occur during construction" but no specific limitations are cited. Duncan (19) proposed that the combination of moment and thrust be controlled so that the safety factor with respect to the formation of a plastic hinge (F_p) would be 1.65 or more, which is designed to avoid the initiation of yielding in the conduit wall. According to the Ontario code, for elliptical and round conduits, the upward crown deflection should not exceed

$$\delta_{crown} = 1{,}000\mu_2 D_H^2/d \tag{5}$$

where

δ_{crown} = upward crown deflection (mm),
μ_2 = shape factor,
D_H = horizontal diameter, (m), and
d = corrugation depth (mm).

For problems in the fifth group, values of μ_2 were back-calculated from Equation 5 by using the maximum elongation in vertical diameter in lieu of δ_{crown}. This substitution was made because the change in vertical diameter is a better measure of the change in shape than δ_{crown}; to reflect the fact that the actual change in shape is only approximated, μ_2 was retermed the "deflection factor." A plot of the calculated values of μ_2 versus F_p is shown in Figure 15. It is seen that the provisions in the preliminary code corresponded, essentially, to $F_p = 1$.

In consideration of the facts that in practice perfect symmetry in the deflected shape of the conduit cannot be maintained, that the effects of loading due to compaction equipment have not been considered in the analysis, and that the rate at which deflections increase as F_p approaches 1 is very high, it was recommended to limit μ_2 to values that correspond to $F_p = 1.05$. A plot of these values of μ_2 versus the ratio of horizontal to vertical diameter (D_H/D_V) is shown in Figure 16. This recommendation was not adopted in the approved code (25), although a provision to limit the difference in backfill heights on the two sides of any transverse section to a maxi- mum of 600 mm (2 ft) was included.

CONCLUSIONS

Results obtained with different soil models examined in this study are very different, especially with respect to deflection and bending moments in the conduit walls. An "equivalent" linear-elastic soil model capable of modeling various phases of soil-conduit interaction does not exist; that is, different moduli would have to be selected when a given factor, say deflection, was considered at different stages in the construction process. Moreover, at any construction stage different moduli are required to predict different response factors, for example, thrust, deflection, or moment. In many instances the errors associated with the use of a single set of linear-elastic soil moduli are very large; accordingly, further use of this soil model for prediction purposes should be abandoned. The overburden-dependent soil model gave similar unrealistic results. The extended Hardin model followed the trends well but generally resulted in a stiffer soil response than that which would be expected from the interpreted level of soil compaction.

The Duncan-Chang soil model [using Young's modulus (E) and Poisson's ratio (μ)] gives a good representation of soil response in the soil-conduit system. However, when the shear strength of the soil

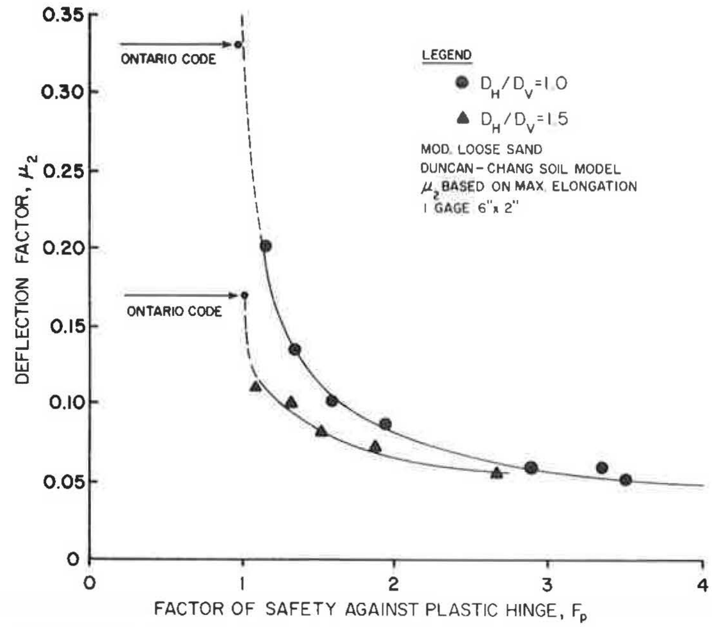

FIGURE 15 Deflection factor versus safety factor.

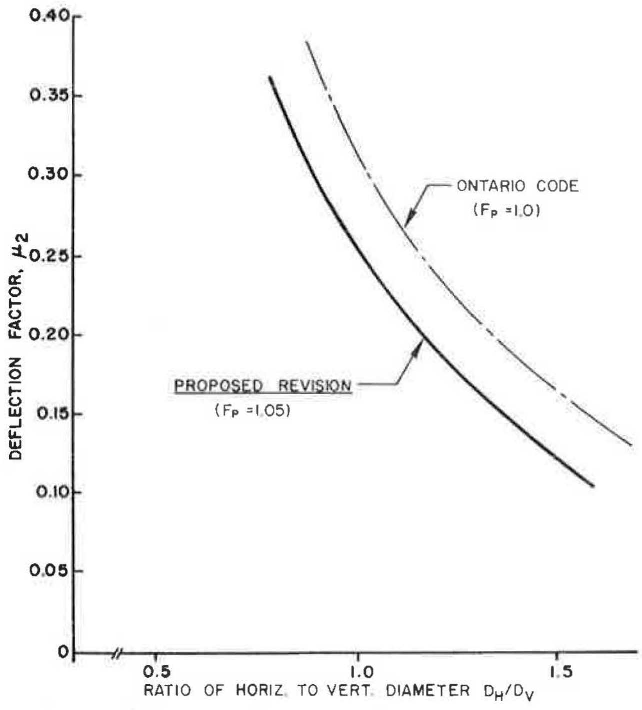

FIGURE 16 Deflection factor versus D_H/D_V.

is approached, the soil response is poorly modeled and arbitrary limitations on the soil moduli must be applied to avoid convergence problems. The modified Duncan model [using Young's modulus (E) and bulk modulus (B)] is similar to the Duncan-Chang model; given comparable limitations to simulate failed elements, the two models are equally resistant to convergence problems although the response of the modified Duncan model is somewhat "softer" than that of the DuncanChang model (3). Because it possesses a large data base and gives results that are compatible with field experience at moderate stress levels, the Duncan-Chang soil model is recommended for routine studies of soilconduit interaction. However, the results should be viewed with caution if inspection of the computer output shows more than three or four failed elements adjacent to the conduit wall.

The response of buried conduits is strongly affected by the soil-conduit interface behavior. Predictions based on results obtained by enforcing the fully bonded interface condition are often unrealistic and should be viewed with great caution, particularly in cases where substantial deflections are anticipated.

The response of long-span buried conduits is sensitive to the sequence of placement of soil layers around the conduit. This emphasizes the need for good construction practice, a need that has long been recognized. When predictions are compared with field measurements, attention must be paid to the details of soil placement if meaningful comparisons are to be made.

Initiation of yielding in the conduit walls during construction is not necessarily detrimental, provided the backfill is granular and moderately well compacted and the distortion of the conduit is not overly asymmetrical. In the absence of buckling, yielding during subsequent placement of soil cover can result in a favorable redistribution of soil pressures, thereby permitting the conduit to support the overburden loads more efficiently. A useful procedure to track the potentially beneficial effects of wall yielding was presented in Figure 9.

Buckling of buried metal conduits is an important failure mode. However, current analytical methods are incapable of estimating the buckling mode, the load at which buckling occurs, or its influence on the subsequent load capacity of the conduit. Although research to elucidate these problems is badly needed, it may well be that the main benefit of rib stiffeners is to guard against snap-through buckling at the crown rather than to provide additional resistance to bending.

A diagram is presented (Figure 14, solid lines) that can be used to predict reliably the maximum thrust in buried circular and elliptical conduits. The diagram is applicable to spans up to 35 ft and depths of soil cover up to 75 ft, provided the backfill around the conduit is granular for at least two pipe diameters. Indications are that arching effects cease to reduce thrusts in the conduit wall significantly once the height of cover above the crown exceeds about 40 ft.

A diagram is presented (Figure 16, solid line) from which the maximum elongation in vertical diameter of circular and elliptical conduits can be estimated. The diagram is applicable to conduits up to 35 ft in diameter and should be useful as a guide to control "peaking" during construction.

The requirements for good predictions of the response of buried conduits, in order of their importance, are

1. Using a nonlinear soil model with appropriate selection of soil parameters;
2. Accounting for slip at the soil-conduit interface and for yielding of the conduit wall, which require iterating each load step to convergence;
3. Starting the analysis when the backfill is at the pipe invert and following the sequence of soil layer placement closely; and
4. For large spans, accounting for self-weight of the structure. In special cases, it may be necessary to use a large strain formulation or to update the geometry after each load step in order to achieve satisfactory results.

Because good predictions have been achieved without accounting for compaction loadings, it is not clear to what extent it is necessary to consider compaction loadings in the analysis.

The modified CANDE code accommodates nonlinear soil properties, automatically generates finite-element meshes for a variety of pipe geometries and boundary conditions, treats any desired slip conditions at the soil-conduit interface and any desired sequence of soil layer placement, and iterates to convergence at each load step up to the point where the wall section reaches a fully plastic hinge. It has demonstrated its versatility and dependability in literally hundreds of computer simulations of a large variety of soil-conduit interaction problems, and good agreement has been obtained with field data in those cases where the predictions could be compared with reliable measurements (5). There is no longer any reason to use simplified (and less reliable) procedures in the analysis of buried metal conduits.

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