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A methodology for evaluating potential damage to cast iron pipes induced by tunnelling

A. Bracegirdle & R.J. Mair
Geotechnical Consulting Group, London, UK

R.J. Nyren
Imperial College of Science, Technology and Medicine, London, UK

R.N. Taylor
Geotechnical Research Centre, City University, London, UK

ABSTRACT: It is often necessary to make simple evaluations of the potential for damage to buried services as a result of settlements caused by tunnelling, without information available on the condition of the services or jointing details. In such cases, assessments can be made on the basis of simple criteria, avoiding the need for sophisticated and time-consuming analyses. This paper describes a methodology that may be used where settlements arise from tunnelling in soft ground, such as London Clay.

1 BACKGROUND

About 80% of buried service pipes in the UK are in grey cast iron. Graphite, present as flakes in the iron, gives the iron its grey colour, and has a marked effect on its properties. The graphite flakes offer little strength and, acting as voids, they provide stress-raisers within the iron. Tensile stresses tend to concentrate around the ends of the flakes, allowing fractures to propagate from flake to flake. As a result, grey cast iron is relatively weak in tension, and is brittle. The modulus of elasticity of grey cast iron in tension is non-linear and rate dependent. In its favour, grey iron may be cast at relatively low temperatures, allowing intricate shapes to be formed; it has good damping properties, and is readily machinable.

As identified by Howe (1985), the most common modes of failure of cast iron pipes subject to ground movements are tensile fracture, and pull-out at joints. The tolerance of individual pipes to ground movements is dependent on many factors, few of which the tunnel designer will have knowledge of. The tensile properties of cast iron can be quite variable, depending on the quality of the casting, and its stress-history. The uniformity of back-fill, corrosion (Harrison, 1976) and the presence of service connections will also affect a pipe's performance. In view of the difficulties of accurate prediction of stresses in pipes, the potential for damage due to ground movement is usually evaluated using empirical correlations, and semi-empirical methods based on strain predictions.

2 A METHODOLOGY

This paper describes a methodology that may be applied to rapidly assess the potential for damage to buried services, where detailed information concerning the services is not immediately available to the designer. The methodology proposed is based on the estimation of ground movements, and comparisons with basic acceptance criteria, as set out in the following steps:

a. compare the predicted ground slope at the level of the pipes with simple empirical criteria;
b. check possible joint rotation and pull-out;
c. calculate potential pipe strains.

The methodology conservatively assumes the worst scenario in terms of the location and stiffness of pipe joints. In steps *a* and *b*, it is conservatively assumed that the pipe follows the ground. In step *b*, no account is taken of the actual location of joints between pipes; it is assumed that joints may be located at the most severe position, and that the joints are free to rotate and slide.

In step *c*, it is assumed that joints are rigid and capable of transferring bending moments. Simple soil-pipe interaction factors, based on elasticity, are used to adjust the predicted ground strains to estimate axial pipe strains. Where tunnels are deep, with wide settlement troughs, predicted pipe strains become particularly onerous; in this case it may become necessary either to obtain more information on the condition, type and frequency of joints, or to incorporate soil-pipe slip into the model.

In the following, the methodology is outlined in greater detail, with particular reference to services parallel or transverse to the line of tunnelling.

3 PREDICTION OF GROUND MOVEMENTS

3.1 *Tunnels*

In soft ground, the transverse settlement trough due to the excavation of a single tunnel can be approximated to a Gaussian distribution; New & O'Reilly (1991) and Mair et al (1996) describe the application of this to the prediction of vertical and horizontal movements due to single and multiple tunnels. The assumption of the longitudinal settlement trough corresponding to a cumulative probability curve, as described by Attewell & Woodman (1982), allows a generalised expression for settlement, S_v, of the form:

$$S_v = \frac{V_s}{\sqrt{2\pi}\ i} \cdot e^{\frac{-y^2}{2i^2}} \cdot \left[G\left(\frac{x-x_i}{i}\right) - G\left(\frac{x-x_f}{i}\right)\right] \qquad \textbf{(1)}$$

where x and y are tunnel coordinates, as shown on Fig. 1, and G is the value of the cumulative normal distribution function.

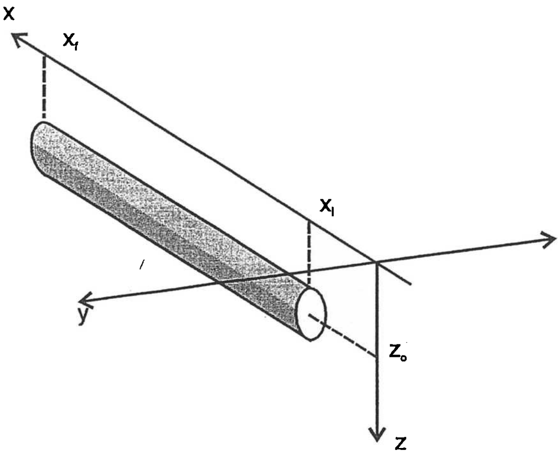

Figure 1: Coordinate system, after Attewell & Woodman (1982)

The volume loss, V_s, can be estimated from field and experimental data, as described by O'Reilly & New (1982) and may be correlated with tunnel stability (Mair, 1979). Construction of tunnels in London Clay typically results in a volume loss of between 1 and 2% and in this case it is usual to assume 2% when assessing the potential for damage to buried services. O'Reilly & New (1982) showed that the trough width parameter, *i*, at the ground surface is about 50% of the depth to the tunnel axis in clay, z_o, and about 25% in sand. The value of *i* at the depth of services, z_p, can be estimated for tunnels in clay, using the following relationship, derived from case studies by Mair et al (1993):

$$i = z_o\left(0.175 + 0.325\left(1 - \frac{z_p}{z_o}\right)\right) \qquad \textbf{(2)}$$

Equation 2 should only be applied when estimating ground movements above the level of the crown of the tunnel.

An alternative method of estimating ground movements close to the tunnel is given by New & Bowers (1994).

The form of typical transverse and longitudinal settlement troughs are shown on Fig. 2. Horizontal ground movements are estimated assuming the vector addition of incremental vertical and horizontal movements in the direction of the centre of the tunnel face. Horizontal movements in a fully developed settlement trough, S_{hf}, above a horizontal tunnel, are therefore towards the tunnel centreline and are given by:

$$S_{hf} = \frac{y}{z_o - z_p} S_v \qquad \textbf{(3)}$$

3.2 *Circular Shafts*

Very few measurements of ground movements have been made around deep circular shafts in stiff clays, possibly because ground movements are generally small and seldom cause problems. Recently, detailed measurements of movements were made around the Heathrow Express trial shaft; the construction of an 11m diameter, 26m deep shaft, and the movement data are described by New & Bowers (1994). Data were collected along two lines of measuring stations. Consistent settlement data were recorded on both lines, although the data for horizontal movement were considered reliable on one line only. Measured horizontal movements, considered to be reliable, were 4mm or less.

The normalised settlement data are reasonably well represented by:

$$\frac{S_v}{H} = \alpha\left(1 - \frac{d}{H}\right)^2 \qquad \textbf{(4)}$$

where S_v is the settlement at a distance d from the shaft wall, H is the depth of the shaft, and α is a dimensionless coefficient. In this case, the best fit was with a value of α of 6×10^{-4}. The above relationship should however be treated with caution, as it relates to one site, and does not depend on shaft diameter. Particular care should be exercised when applying the relationship to shafts of more than 11m in diameter.

3.3 *Complex Geometries*

Where multiple tunnels are to be constructed, or where other activities, such as deep excavation or shaft sinking, are to be undertaken, the predicted ground movements should be superimposed to generate the most potentially damaging combination of ground movements affecting the services. Ground strains may be obtained directly by differentiating the displacements, or from a Gaussian curve fitted to the predicted ground movements; curve fitting offers the advantage of application of simple empirical criteria for assessment of potential damage.

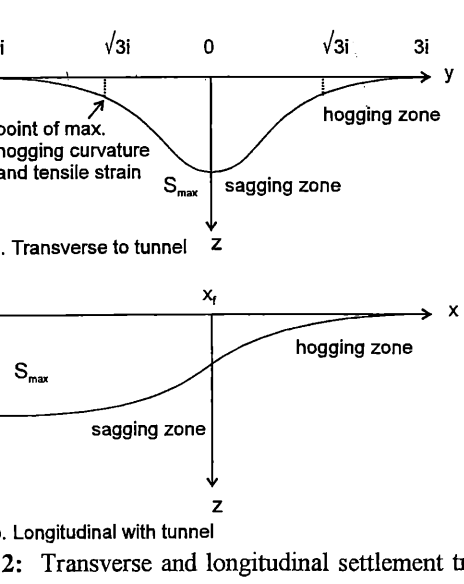

Figure 2: Transverse and longitudinal settlement troughs

4 EMPIRICAL CRITERIA

O'Rourke & Trautman (1982) provide an empirical method for a preliminary evaluation of potential pipe damage based on observed values of slope at which damage occurs. The relationship of the ratio S_{max}/i with observed damage is a function of the assumed Gaussian distribution of settlement, and is based on settlement observations for relatively shallow tunnels in sand; they propose the limits for S_{max}/i for a transverse settlement trough shown in Table 1.

Table 1: Empirical criteria

Description of pipe	S_{max}/i limit
relatively rigid pipes, more than 200mm diameter	0.012 (1:140 slope)
relatively flexible pipes, less than 200mm diameter	0.012-0.040 (1:40-140 slope)

The criteria assume that the pipes being considered are not leaking prior to tunnelling, nor do they have a history of repeated leakage; where pipes have a history of leakage, it must be assumed that any ground movement will exacerbate leakage.

Figure 3 shows the application of the criteria to a single tunnel in clay, having 2% volume loss, and assuming the pipes are transverse to the tunnel at a depth of 1m. As can be seen from Fig. 3, the criteria predict little risk of damage, where tunnels are less than 5m in diameter, and the depth to the tunnel axis is greater than 8m.

The empirical criteria proposed by O'Rourke & Trautman provide a useful preliminary means of assessing the likelihood of damage to services. They do not, however, reflect variation in risk with the condition and type of utility; for example, no allowance is made for the risk associated with very sensitive services, such as large diameter gas pipes in grey cast iron, where the consequences of rupture may be very severe. It is

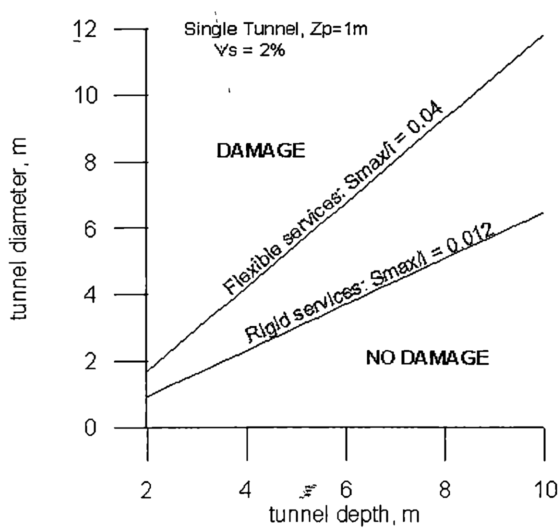

Figure 3: Empirical criteria, after O'Rourke and Trautman (1982)

recommended that joint rotation and pull-out, and pipe strains are calculated where gas or high pressure water mains are present, and that these be compared with acceptance criteria, discussed in Section 5.

5 JOINT ROTATION AND PULL-OUT

Attewell et al (1986) give guidance for cast iron mains subjected to ground movements, as shown in Table 2.

Table 2: Allowable joint rotation and pull-out

Description	Rotation, θ, (degrees)	Pull-out, R, (mm)
lead-yarn joint in gas main with history of leaks	none	none
lead-yarn joint in sound gas main	1.0	10
lead-yarn joint in water mains	1.5	15
rubber gasket joint in gas or water mains	2.5	25

Where there is sufficient information, or in the case of particularly sensitive services, individual joints may be considered. Alternatively, it may be conservatively assumed that joint rotation, θ, is given by:

$$\theta = 2\ \tan^{-1}\left(\frac{S_{v\ max}}{\sqrt{2\pi}\ i}\right) \tag{5}$$

where pipes are transverse to the tunnel line, or,

$$\theta = \tan^{-1}\left(0.4\ \frac{S_{v\ max}}{i}\right) \tag{6}$$

where pipes are parallel to the tunnel line. The above relationships assume the worst conceivable configuration of joints, and in some instances may be excessively conservative.

Again conservatively, it may be assumed that the maximum potential pull-out is equal to the maximum predicted horizontal ground displacement due to tunnelling for both longitudinal and transverse directions. Examples of the application of joint rotation and pull-out criteria for gas mains in sound condition above tunnels in clay are given in Figs. 4 and 5; in this case, the criteria are broadly similar to those of O'Rourke and Trautman, shown in Fig. 3.

6 PIPE STRAINS

6.1 *Allowable Pipe Strains*

As outlined in Section 1, cast iron pipes are sensitive to increased tensile strain caused by tunnelling. In general, compressive strains are less critical. The total tensile strain

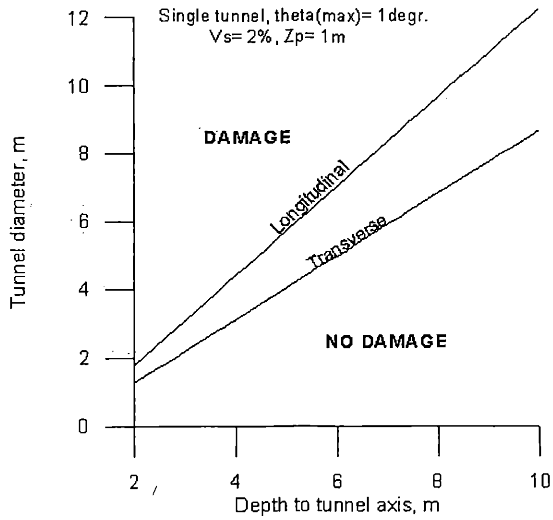

Figure 4: Potential damage due to joint rotation, $\theta_{max} = 1°$

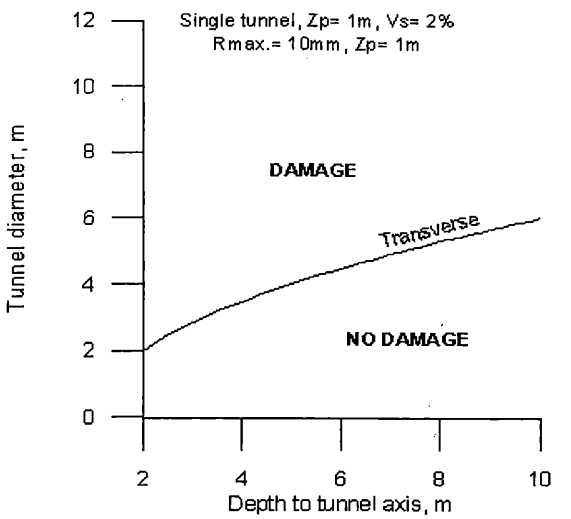

Figure 5: Potential damage due to pull-out, $R_{max} = 10mm$

arises from two components: bending strain due to curvature, and axial strain. Published values of strain at rupture of cast iron vary considerably, depending on the type of test and quality of casting; values of between 4000 and 6000με are not uncommon. In practice, however, imperfections in casting can act as stress raisers, and failure may take place in thin sections at tensile strains of 2000με (Bracegirdle et al, 1996). In addition, deterioration of cast iron may occur following overstress arising from previous ground movement (Howe, 1985) and related corrosion (Harrison, 1976). As a result design codes have been relatively cautious; Attewell et al (1986) provide a summary of total allowable strains arising from code limitations on direct tensile stress, used in design and shown in Table 3.

Some conservatism is also required when assessing acceptable limits for additional strain due to ground movements in otherwise sound pipes. The criteria given in Table 4 are recommended for application to the simple methodology described in this paper.

Table 3: Typical design strain

Material	Design strain (με)	
	tensile	compressive
pit cast grey iron	370	1550
spun cast grey iron	430-490	1770-2040
ductile iron	820	1020

Table 4: Recommended allowable increase in strain

Material	Allowable strain με	
	tensile	compressive
pit cast and spun cast grey iron	100	1200
ductile iron	500	700

The criteria in Table 4 are conservative; Herbert and Leach (1990) state that it is reasonable to allow up to 200με for pipes in grey iron exceeding 300mm in diameter and 150με for smaller pipes; values of 150 and 100με respectively being applied in particularly adverse situations. Simple procedures for estimating pipe strains are summarised by Attewell et al (1986); the methodology described in Section 6.2 draws on these procedures, but with some simplifications.

6.2 *Estimation of Pipe Strain*

As discussed in Section 2, ground movements above tunnels may be estimated assuming a Gaussian distribution of surface settlement. *Transverse* to the line of a single tunnel, the maximum curvature, d^2S_v/dy^2, occurs in sagging, at the centre of the trough, where:

$$\frac{d^2 S_{v\,(y=0)}}{dy^2} = \frac{S_{v\,max}}{i^2} \qquad (7)$$

Large compressive horizontal ground strains occur at the centre of the trough. As shown in Fig. 2, the maximum hogging curvature and horizontal tensile strain occurs at a distance of $\sqrt{3}$ i from the centre of the trough (equations 8 & 9); under most circumstances, the combination of axial and bending strains produce the greatest tensile strains in pipes at this point:

$$\frac{d^2 S_{v(y=\sqrt{3}i)}}{dy^2} = 0.446\,\frac{S_{v\,max}}{i^2} \qquad (8)$$

Horizontal ground strains are given by the expression:

$$\frac{dS_{h\,(y=\sqrt{3}i)}}{dy} = 0.446\,\frac{S_{v\,max}}{z_o - z_p} \qquad (9)$$

where S_h is horizontal ground movement. Similar expressions may be derived for ground strains directly above and in line with the tunnel (for example Rankin, 1988). In the *longitudinal* direction maximum curvature and horizontal strains are given by:

$$\frac{d^2 S_{v\,(x_f \pm i)}}{dx^2} = 0.242\,\frac{S_{v\,\max}}{i^2} \qquad \textbf{(10)}$$

and

$$\frac{dS_{h\,(x_f\ i)}}{dx} = 0.242\,\frac{S_{v\,\max}}{z_o - z_p} \qquad \textbf{(11)}$$

Figs 6 and 7 have been prepared for a 200mm diameter pipe, above a tunnel in clay, assuming pipe strains to be identical to strains in the ground; as can be seen, this assumption is particularly onerous when considering axial strains. In addition, it can be seen that bending strains in pipes transverse to tunnels tend to be more severe than for pipes runing parallel to tunnels.

Attewell et al (1986) provide simple procedures for calculating strain reduction factors, for pipe curvature and for axial strain, which make allowance for soil-pipe interaction. In the simple methodology proposed in this paper, it is assumed that pipes follow ground movements when calculating bending strain; the maximum horizontal tensile ground strain is modified by a strain reduction factor to give the maximum axial pipe strain; pipe joints are conservatively assumed to be rigid. The reduction factor is based on elasticity, making no allowance for slip between the pipe and soil; analyses incorporating slip have shown this approach to be quite conservative. To determine the reduction factor, RF, the following parameters must be determined or estimated:

E_p	elastic modulus of pipe material
E_g	elastic modulus of soil surrounding pipe
A_p	cross sectional area of pipe $(=\pi(2r_p\text{-}t)t)$
R_a	pipe area ratio $(=A_p/\pi r_p^{\,2})$
K^*	soil-pipe stiffness factor $(= E_pR_a/E_g)$

On Figs. 8 and 9, the reduction factor is plotted for varying K^*, against d/i, where d is the diameter of the pipe. The appropriate reduction factor can be obtained from either of Figs. 8 or 9, depending on whether pipes lie transverse or parallel to the tunnel. Axial pipe strain transverse to the tunnel, ϵ_{py}, for example, is given by:

$$\epsilon_{py} = RF.\ \frac{dS_{hy}}{dy} \qquad \textbf{(12)}$$

Conservative estimates of combined tensile strain due to bending and axial ground movement are obtained by combining the strain due to pipe curvature, assuming the pipe moves with the ground, and maximum tensile ground strain modified by the reduction factor. If the predicted strains are in excess of the proposed limits on strain, given in Table 4, the following options are available:

a. undertake more sophisticated modelling, incorporating soil-pipe slip, etc (eg. Leach, 1984);

b. expose the pipes, determine the condition and nature of joints, and re-analyse;

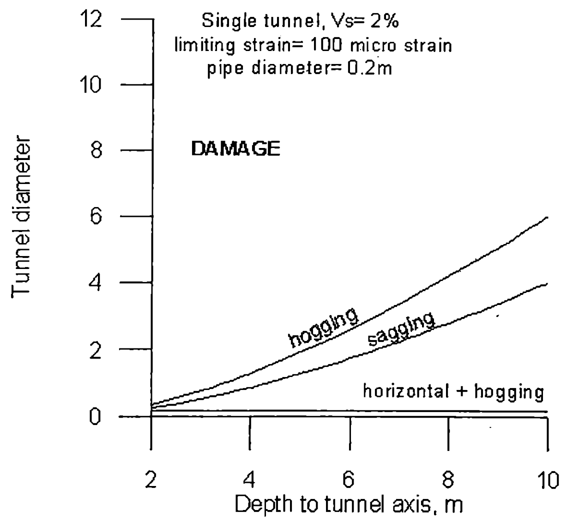

Figure 6: Pipe strains transverse to tunnel, assuming pipe follows ground

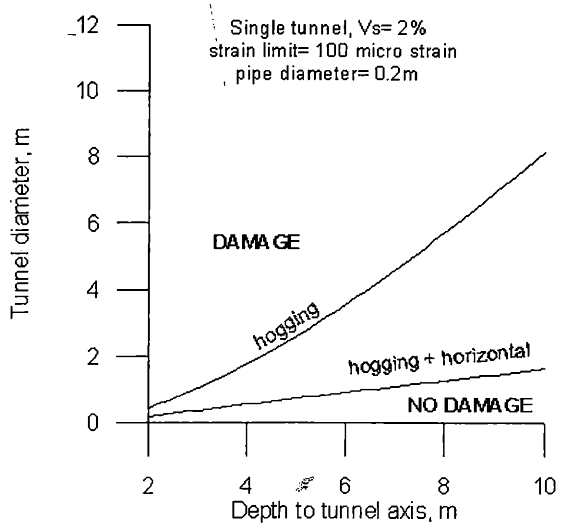

Figure 7: Pipe strains longitudinal with tunnel, assuming pipe follows ground.

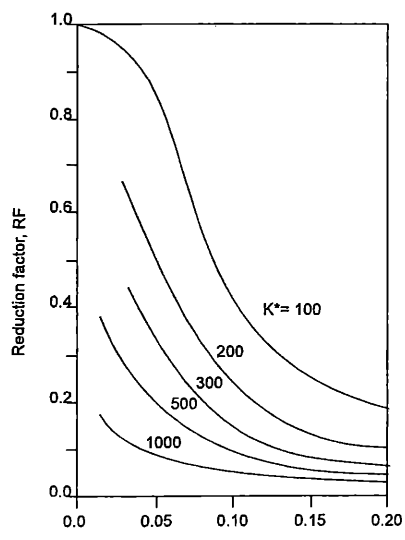

Figure 8: Reduction Factor for transverse pipes (after Attewell et al, 1986)

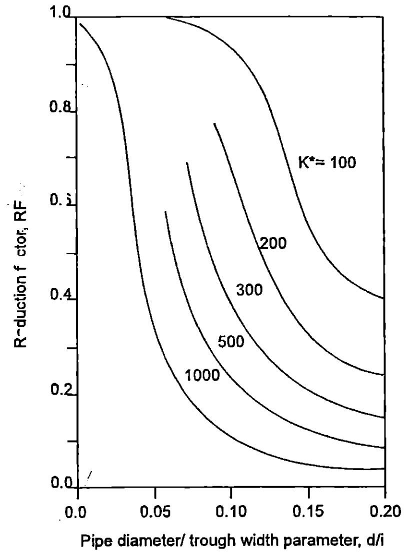

Figure 9: Reduction Factor for longitudinal Pipes (after Attewell et al, 1986)

c. undertake preventative works, such as diversion, ground isolation or compensation grouting beneath the pipes.

Additional analysis may be necessary where service connections are located in critical positions relative to the tunnels. In this case a similar approach to that described in this paper may be adopted; the connections are firstly assumed to be flexible (steps *a* & *b*) and then rigid (step *c*). Pipes may be considered as simple elastic elements and treated using solutions for piles (for example Poulos & Davis, 1980, and Davisson, 1970).

7 SUMMARY

The principal concern when considering the effects of tunnelling on buried services is frequently the brittle behaviour of cast iron pipes, and tensile strains induced by tunnelling. The information available on the pipes and jointing is often limited. A simplified conservative methodology is proposed, in which a series of steps is taken, assuming either flexible or rigid jointing of pipes, and comparisons made with various acceptability criteria. If the criteria are violated, more detailed analysis may be required, or steps should be taken to mitigate the potential for damage.

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