5

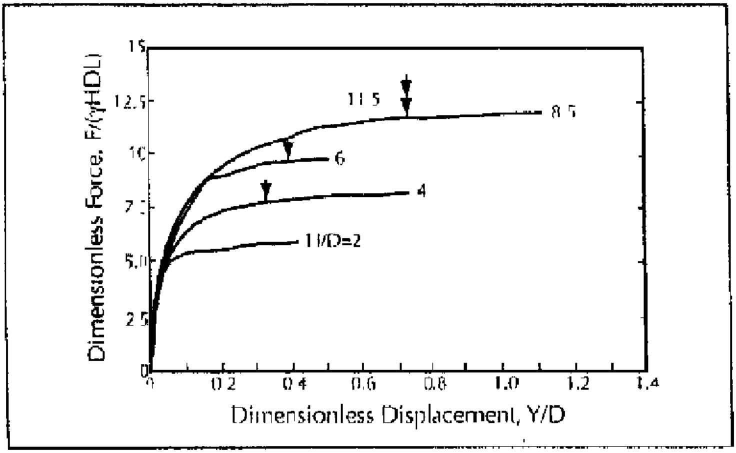
Soil-Pipe Interaction

Buried pipelines are damaged in earthquakes due to forces and deformation imposed on them through interactions at the pipe-soil interface. That is, the ground moves and thereby causes the pipe to deform. For purposes of analysis, any arbitrary ground deformation can be decomposed into a longitudinal component (soil movement parallel to the pipe axis) and a transverse component (soil movement perpendicular to the pipe axis). Both those types of pipe-soil interactions are discussed in this chapter. In the transverse direction, interaction involves relative deformation and loading in both the horizontal and vertical planes. For relative ground movement in the vertical direction, one must distinguish between upward and downward pipe movement since the interaction forces are different for these two cases. Finally one must distinguish between pipe surrounded by competent, non-liquefied soil, and pipelines located in a liquefied layer.

5.1 Competent Non-Liquefied Soil

Soil interaction forces for a pipeline surrounded by competent, non-liquefied soil are well established. They are based upon laboratory tests. For example, Trautmann and T O'Rourke (1983) established a force-deformation relation for horizontal lateral movement as shown in Figure 5.1.

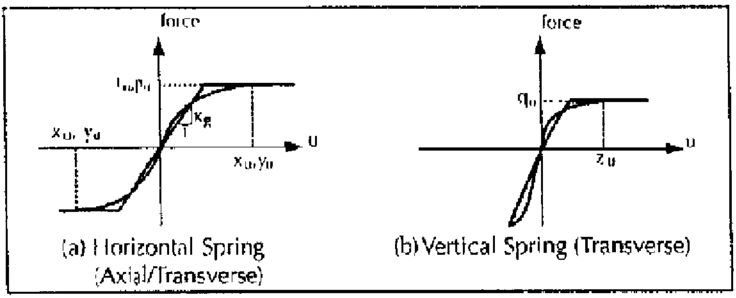
The ASCE Technical Council on Lifeline Earthquake Engineering (TCLEE) Committee on Gas and Liquid Fuel Lifelines (ASCE, 1984) have suggested, for the purpose of analysis, idealized elasto-plastic models as shown in Figure 5.2. Note that the elasto-plastic model is fully characterized by two parameters; the maximum re-



After Trautmann and O'Rourke 1983

■ **Figure 5.1 Lateral Load-Deformation Relation at Pipe-Soil Interface**

sistance t_u, p_u, q_u, in the horizontal axial, horizontal transverse and vertical transverse directions respectively, having units of force per unit length and the maximum elastic deformation x_u, y_u, z_u, having units of distance. The equivalent elastic soil spring coefficients, having units of force per unit area, is simply the ratio of the maximum resistance divided by a half of the maximum elastic deformation, for example $2t_u/x_u$ for the horizontal axial (longitudinal) case. Note that this spring coefficient is effective only for relative displacements less than the maximum values of x_u, y_u, z_u after which the resistance is constant.



After ASCE, 1984

■ **Figure 5.2 Idealized Load-Deformation Relations at Pipe-soil Interface**

5.1.1 Longitudinal Movement

Relative movement parallel to the pipe axis results in longitudinal (horizontal axial) forces at the pipe-soil interface. For the elasto-plastic model, the ASCE guideline provides the following relations for clay and sand.

For sand,

$$t_u = \frac{\pi}{2} D\bar{\gamma} H(1 + k_o)tank\phi \tag{5.1}$$

$$x_u = (0.1 \sim 0.2) \text{ in} = 2.54 \sim 5.08\times10^{-3} \text{ m} \tag{5.2}$$

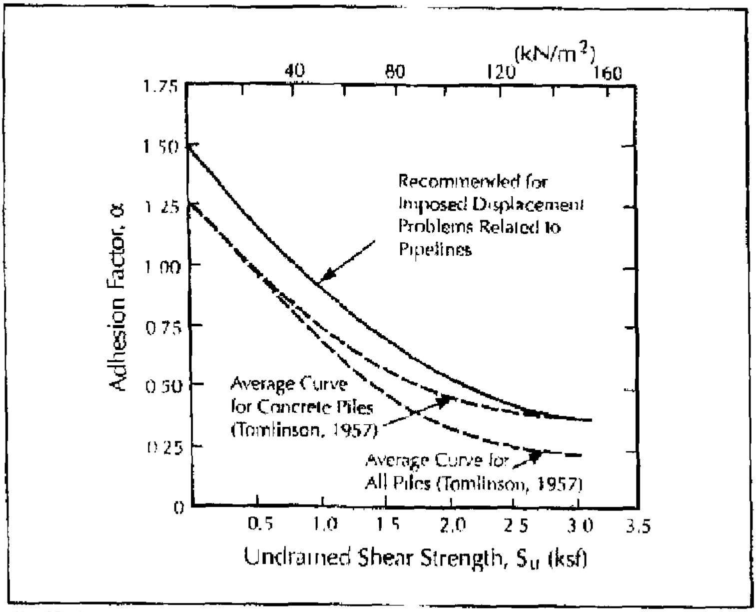
For clay,

$$t_u = \pi D\alpha S_u \tag{5.3}$$

$$x_u = (0.2 \sim 0.4) \text{ in} = 5.08 \sim 10.16\times10^{-3} \text{ m} \tag{5.4}$$

where D is the pipe diameter, S_u is the undrained shear strength of the surrounding soil, α is an empirical adhesion coefficient varying with S_u, $\bar{\gamma}$ is the effective unit weight of the soil, H is the depth to center-line of the pipeline, ϕ is the angle of shear resistance of the sand and k_o is the coefficient of lateral soil pressure at rest. The magnitude of k_o for normally consolidated cohesionless soil has been reported to range from 0.35 to 0.47. However, one expects k_o to be somewhat larger because of the backfilling and compaction of the soil around pipelines. T. O'Rourke et al. (1985) recommend that k_o=1.0, as a conservative estimate under most conditions of pipeline burial. Finally, k is the reduction factor depending on the outer-surface characteristics and hardness of the pipe. For a concrete pipe, steel or cast iron pipe with cement coating, k=1.0, for cast iron or rough steel, k ranges from 0.7 to 1.0, while for smooth steel or for a pipe with smooth, relatively hard coating, k ranges from 0.5 to 0.7.

The ASCE guideline recommends the relationship between α and S_u shown in Figure 5.3, in which the adhesion factor is a decreasing function of the undrained shear strength of the soil



After ASCE 1984

■ **Figure 5.3 Adhesion Factors vs. Undrained Shear Strength**

5.1.2 HORIZONTAL TRANSVERSE MOVEMENT

Relative movement perpendicular to the pipe axis in the horizontal plane results in horizontal transverse forces at the pipe-soil interface. For the elasto-plastic model, the ASCE guideline provides the following relations for sand and clay.

For sand,

$$p_u = \bar{\gamma} H N_{qh} D \tag{5.5}$$

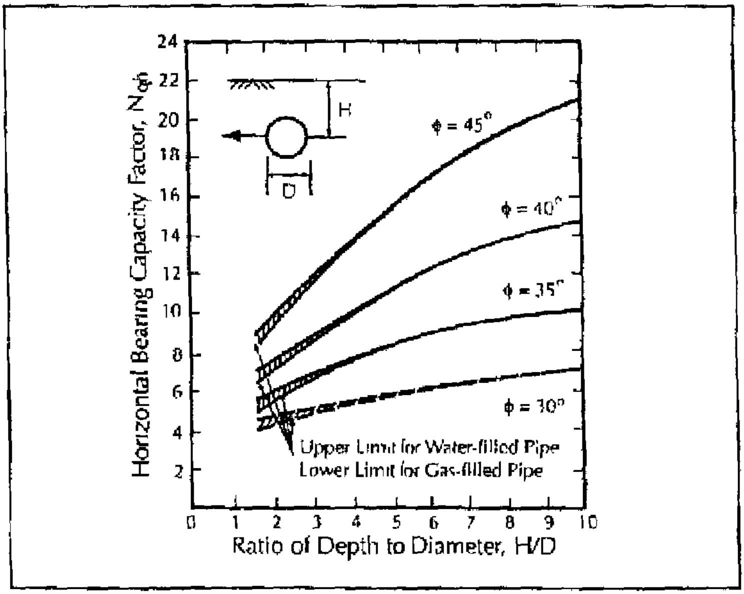
$$y_u = \begin{cases} (0.07 - 0.10)(H + D/2) & \text{for loose sand} \\ (0.03 - 0.05)(H + D/2) & \text{for medium sand} \\ (0.02 - 0.03)(H + D/2) & \text{for dense sand} \end{cases} \tag{5.6}$$

For clay,

$$p_u = S_u N_{ch} D \quad (5.7)$$

$$y_u = (0.03 \sim 0.05)(H + D/2) \quad (5.8)$$

where N_{qh}, N_{ch} are the horizontal bearing capacity factors for sand and clay, respectively. N_{qh} is shown in Figure 5.4 for sand N_{ch} is equivalent to N_{qh} with ϕ=30°.



After Trautmann and T O'Rourke, 1983

Figure 5.4 Horizontal Bearing Factor for Sand vs Depth to Diameter Ratio

5.1.3 Vertical Transverse Movement, Upward Direction

Relative upward movement perpendicular to the pipe axis results in lateral forces at the pipe-soil interface. For the elasto-plastic model, the ASCE guideline provides the following relations for clay and sand.

For sand,

$$q_u = \bar{\gamma} H N_{qv} D \tag{5.9}$$

$$z_u = (0.01 \sim 0.015)H \tag{5.10}$$

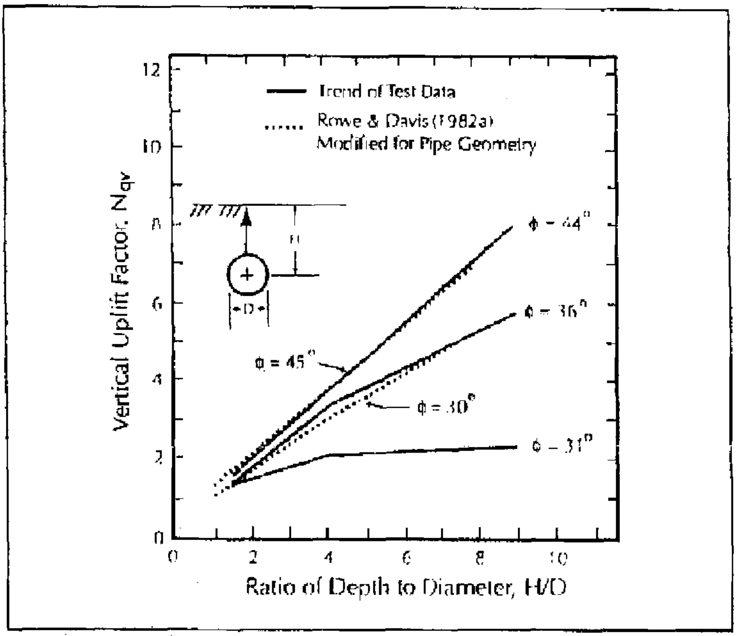
For clay,

$$q_u = S_u N_{cv} D \tag{5.11}$$

$$z_u = (0.1 \sim 0.2)H \tag{5.12}$$

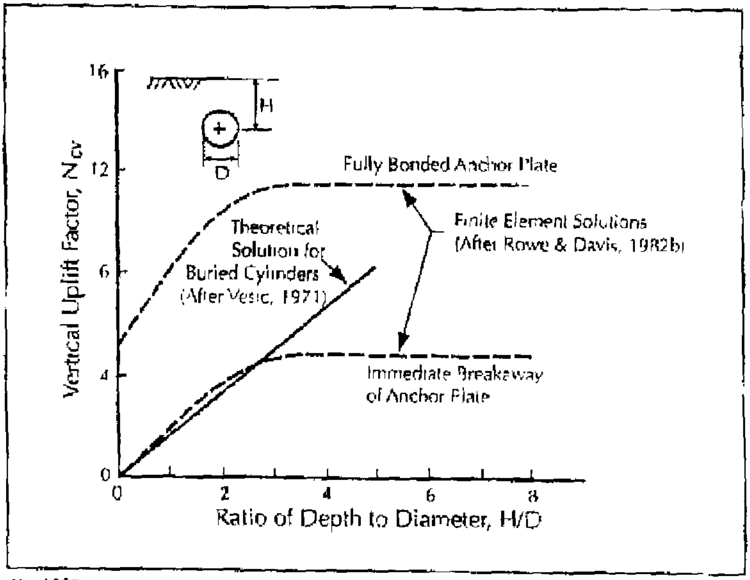
where N_{qv} is the vertical uplift factor for sand and N_{cv} is the vertical uplift factor for clay.

N_{qv} and N_{cv} are presented as functions of the depth over diameter ratio in Figure 5.5 and Figure 5.6, respectively.



After Trautmann and T.O'Rourke 1983

■ Figure 5.5 Vertical Uplift Factor for Sand vs. Depth to Diameter Ratio



After ASCE 1984

Figure 5.6 Vertical Uplift Factor for Clay vs. Depth to Diameter Ratio

5.1.4 Vertical Transverse Movement, Downward Direction

Relative downward movement perpendicular to the pipe axis in the vertical plane results in lateral forces at the pipe-soil interface. For the elasto-plastic model, the ASCE guideline provides the following relations for sand and clay.

For sand,

$$q_u = \bar{\gamma} H N_q D + \frac{1}{2}\gamma D^2 N_\gamma \qquad (5.13)$$

$$z_u = (0.10 \sim 0.15)D \qquad (5.14)$$

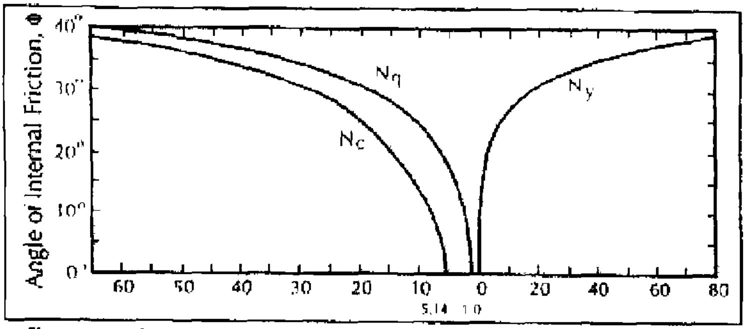
For clay,

$$q_u = S_u N_c D \qquad (5.15)$$

$$z_u = (0.10 \sim 0.15)D \qquad (5.16)$$

where γ is the total unit weight of sand, N_q and N_γ are the bearing capacity factors for horizontal strip footings on sand loaded in the vertically downward direction, while N_c is the bearing capacity factor for horizontal strip footings on clay

The ASCE guideline suggests that these three factors can be obtained from Figure 5.7



■ Figure 5.7 Vertical Bearing Capacity Factors vs. Soil Friction Angle

5.2 Equivalent Stiffness of Soil Springs

As noted previously, interaction at the soil-pipe interface can be modeled as an elastic spring as long as the relative displacement is less than the maximum elastic deformation x_u, etc. In such cases, pipeline response can be determined by "Beam on Elastic Foundation" types of analysis. However, these analyses apply only for small to moderate levels of ground deformation since the maximum elastic deformation (x_u, etc.) typically are small. For example, for a 0.3 m diameter pipe (12 in) in moderately dense sand (γ=1.8×10^{-3} kgf/cm^3, 112 pcf, ϕ=35°) with H=1.2 m (3.9 ft), the maximum elastic deformation for longitudinal, transverse horizontal, transverse upwards and transverse downward are 3.8×10^{-3} m (0.15 in), 0.04 m (1.6 in), 0.018 m (0.71 in) and 0.038 m (1.5 in), respectively

In the following subsection, alternate relations for these spring coefficients are compared with those from the ASCE guideline.

5.2.1 AXIAL MOVEMENT

For axial soil spring constant, the *Specifications for Seismic Design of High Pressure Gas Pipelines* (Japan, 1982) suggests that the soil spring constant is proportional to pipe diameter. M. O'Rourke and Wang (1978) suggest that the soil spring constant is twice the effective shear modulus of soil. Table 5.1 provides a comparison of these three approaches for a pipe with diameter of 0.15 (6 in), 0.30 (12 in) and 0.61 m (24 in) in moderate dense sand (ϕ=35°, γ=1.8×10^{-3} kgf/cm^3, 112 pcf) and burial depth of 1.2 m (3.9 ft).

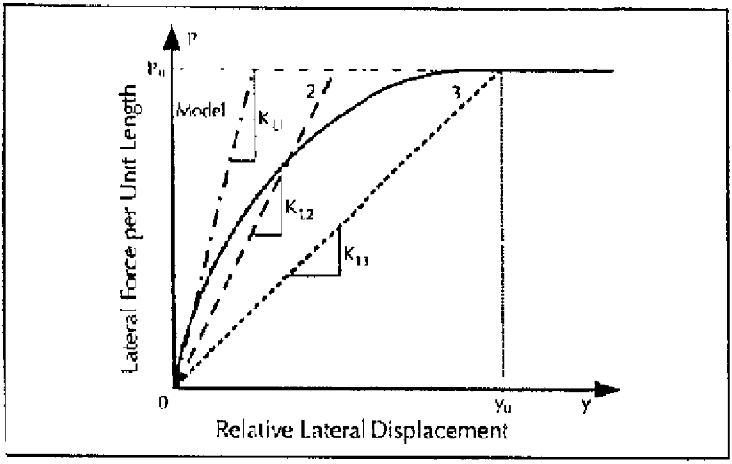
Table 5.1 Comparison of Axial Soil Spring Stiffness

Source	Formula	Stiffness (kgf/cm²)			Note
		D=6 in	D=12 in	D=24 in	
Japanese Gas Association	$\pi D K_n$	28.3	56.5	113	$K_n = 0.6$ kgf/cm^3
O'Rourke and Wang	$2G_s$	60.2	60.2	60.2	$G_s = 66.3\sqrt{\gamma H \frac{1+2k_0}{3}}$
ASCE Guideline (Equivalent)	$\frac{f_w}{x_u/2}$	27.9	59.1	131	$x_u = 3.8\times10^{-3}$ m, k_0=0.6, k=0.9, parameters for Eq 5.[illegible]

As shown in Table 5.1, all three approaches match reasonably well for a 12-inch diameter pipe. However, because the M. O'Rourke and Wang approach does not depend on pipe diameter, it apparently underestimates the axial stiffness for a large-diameter pipe and overestimates the axial stiffness for a small-diameter pipe. The equivalent axial soil spring stiffness from the ASCE guideline matches well with that by the Japanese Gas Association for all three pipe sizes.

5.2.2 LATERAL MOVEMENT IN THE HORIZONTAL PLANE

For lateral movement in the horizontal plane, Audibert and Nyman (1977) proposed three soil spring constants as shown in



After Audibert and Nyman, 1977

■ **Figure 5.8 Soil Spring Constants Corresponding to Different Relative Movement**

Figure 5.8, for modeling the pipe-soil interaction as an elasto-plastic system. That is, K_{l1} and K_{l2} are for small and moderate relative displacements at the pipe-soil interface, respectively. K_{l3} is for relative displacements equal or large than y_u.

Similarly, Thomas (1978) suggests the following soil spring constant, K_l, for large lateral ground movement:

$$K_l = 2.7 \frac{p_u}{y_u} \tag{5.17}$$

For small ground movement such as the movement induced by wave propagation, El Hmadi and M. O'Rourke (1989) suggest the following soil spring constant:

$$K_l = 6.67 \frac{p_u}{y_u} \tag{5.18}$$

Note that these two constants are based on same interaction curve recommended by the ASCE guideline. Equation 5.18 corresponds to the initial slope of the interaction curve, K_{l1}, in Figure 5.8. Hence, this equation conservatively estimates the interaction force even for the wave propagation case.

5.2.3 VERTICAL MUVEMENT

Considering a infinite beam on an elastic subgrade, Vesic (1961) developed a soil spring constant using the Winkler hypothesis for downward pipe movement. The resulting vertical spring stiffness is:

$$K_v = 0.65\left(\frac{E_s D^4}{E_p I_p}\right)^{\frac{1}{12}} \cdot \frac{E_s}{1-\mu_s^2} \qquad (5.19)$$

where E_s is the soil modulus, $E_s = 2(1+\mu_s)G_s$, μ_s is the Poisson ratio of soil, G_s is the shear modulus of soil, $E_p I_p$ is the flexural rigidity of the pipe.

For a steel pipe with a diameter of 30 cm (12 in) and a wall thickness of 0.76 cm (0.3 in), the vertical stiffness from Equation 5.19 ranges from 40 kgf/cm² to 260 kgf/cm² for G_s varying from 32 kgf/cm² to 600 kgf/cm². Using Equations 5.13 and 5.14 and assuming a moderately dense sand (ϕ=35°, γ=1.8×10⁻³ kgf/cm³, 112 pcf) and burial depth of 1.2 m (3.9 ft), the ASCE guideline evaluates a soil spring constant of 140 kgf/cm² (2.0 kips/in²). Note that this value is about an average of the value from Equation 5.19.

5.3 LIQUEFIED SOIL

For a pipeline located in a liquefied layer as opposed to a competent layer, Suzuki et al. (1988) and Miyajima and Kitaura (1989) have shown that the pipe response is very sensitive to the stiffness of the equivalent soil springs. This subsection will discuss the equivalent stiffness of soil springs for a pipe in liquefied soil.

Combining experimental data with analytical solutions based on a beam on an elastic foundation approach, Takada et al. (1987) developed an equivalent soil spring for a pipe in a liquefied soil. They indicate that the equivalent stiffness ranges from 1/1000 to 1/3000 of that for non-liquefied soil. On the other hand, Yoshida and Uematsu (1978), Matsumoto et al. (1987), Yasuda et al. (1987), and Tanabe (1988) suggest that the stiffness ranges from 1/100 to 3/100 of that for non-liquefied soil based on their model experiments.

Miyajima and Kitaura (1991) also conducted model tests which indicated that the stiffness is related to the effective stress in the liquefied soil. That is, the soil spring constant is an increasing function of effective stress and hence, a decreasing function of excess pore water pressure ratio.

For saturated sandy soil, T. O'Rourke et al. (1994) proposed a reduction factor, R_f, for a pipe or pile subject to transverse ground displacement, as:

$$R_f = \frac{N_{qh}}{K_c} \frac{1}{0.0055(N_1)_{60}} \tag{5.20}$$

where K_c is the bearing capacity factor for undrained soil and $(N_1)_{60}$ is the corrected SPT value.

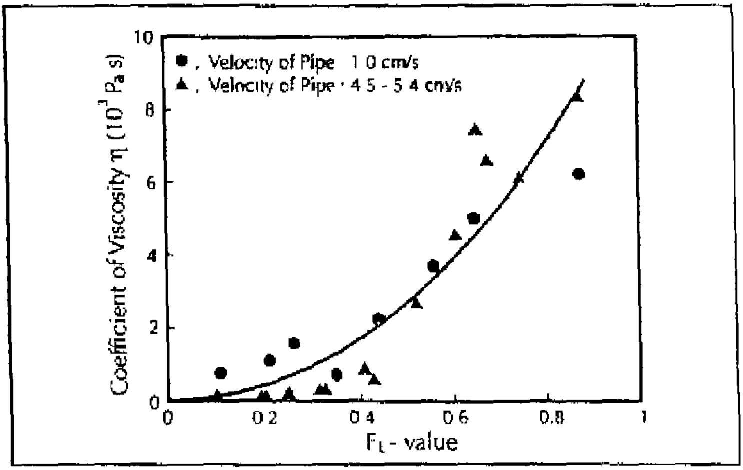
The reduced stiffness at the pipe-soil interface is then given by the stiffness for non-liquefied soil divided by the reduction factor. Their results suggest that the equivalent stiffness ranges from 1/100 to 5/100 of that for non-liquefied soil. Hence, both the longitudinal and transverse stiffnesses for a pipe in a liquefied soil can be taken as about 3% of the stiffness for a pipe in competent soil.

For the approach described above, the liquefied soil is treated, more or less, as a very soft solid. A liquefied soil can also be viewed as a viscous fluid. For that model, the interaction force at the pipe-soil interface varies with the relative velocity between the pipe and surrounding soil. According to Sato et al. (1994), the transverse force imposing on the pipe per unit length is:

$$F = \frac{4\pi\eta V}{2.002 - \log R_e} \tag{5.21}$$

where η is the coefficient of viscosity for the liquefied soil, V is the velocity of the pipe with respect to the liquefied soil, $R_e = \rho VD/\eta$, is the Reynolds number and ρ is the density of liquefied soil.

Based on model tests, Sato et al. (1994) established a relation between the coefficient of viscosity and the liquefaction intensity factor, F_l. This relation is shown in Figure 5.9.



After Sato et al., 1994

■ Figure 5 9 Coefficient of Viscosity vs. F_L-Value

Japanese Road Association (1990) defined the factor of liquefaction intensity, F_L, as:

$$F_L = \frac{0.0042 D_r}{(a_{max} / g)\ (\sigma_v / \sigma'_v)} \tag{5.22}$$

where D_r is the relative density of the soil, a_{max} is the maximum acceleration of the ground, σ_u is the total overburden pressure and σ'_u is the effective overburden pressure.

Note that there are two problems with modeling liquified soil as a fluid. First of all, the velocity of the soil, which is an upper bound for the velocity of the soil with respect to the pipeline, typically is unknown. Secondly, when the liquefied soil stops flowing (i.e., V = 0), Equation 5.21 suggests that there would be no restoring spring force at the soil-pipeline interface which seems counterintuitive.