Seismic Influence on Subsea Pipeline Stresses

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Abstract

The safety analysis of an earthquake is carried out during the operation of a subsea pipeline and an onshore pipeline. Several cases are proposed for consideration. In the case of a buried pipeline, permanent ground deformation by the earthquake and an increase of internal pressure by the acceleration of the earthquake should be considered. In the case of a subsea pipeline, a bending moment is caused by liquefaction of the backfill material on a trenched seabed, etc., which results in a high bending moment of the buried pipeline. The bending moment causes the collapse of the subsea pipeline or a leak of crude oil or gas, which results in economic loss due to enormous environmental contamination and social economic loss owing to operation functional failure. Thus, in order to prevent economic loss and operation loss, structurally sensitive design with regard to seismic characteristics must be performed in the buried pipeline in advance, and the negative impact on the buried pipeline must be minimized by conducting a thorough analysis on the seabed and backfilling material selection. Moreover, it is proposed to consider the selection of material properties for the buried pipeline. A more economical review is also required for detailed study.

Keywords: Natural period of surface layer, Shear wave velocity, Damping ratio for soil structure, Risk level factor, Ground coefficient, Soil stiffness

1. Introduction

If buckling or the failure of subsea pipelines occurs owing to an earthquake, this can have a disastrous effect on the ocean environment and on financial issues because of the leakage of oil, etc. Therefore, research into subsea pipelines must consider the permanent deformation of the ground owing to an earthquake, deformation by liquefaction of the soil, deformation by faults, deformation by the propagation of the earthquake, and deformation by pressure during the operation of subsea pipelines. Seismic waves can cause a small strain on a subsea pipeline. It is important to check the stability of subsea pipelines against earthquakes. The load or strain caused by seismic waves does not significantly affect the initial design, but it can cause strain on the buried pipeline. This strain tends to be mainly within the breaking stress of the material, and is very small because it has no inertial force or a very small inertial force from the dynamic loads. Nevertheless, seismic waves can sometimes damage unburied pipelines, connections between buried and unburied pipelines, connections between subsea platforms and pipelines, etc. Thus, in this study, the influencing factors on the pipeline were considered. These factors are listed in Tables 1.1 and 1.2 in terms of the axial and horizontal strains on the pipelines caused by internal hydrodynamic...
Table 1.1 Classification of seismic characteristics for submarine pipeline

| Classification | Pressure (kgf/cm²) |
|----------------|-------------------|
| I              | ≥10               |
| II             | 3 < P < 10        |
| III            | ≤3                |

Table 1.2 Important factors for different classes of pipeline (Iₚ)

| Class of pipeline | Wave propagation | Faulting | Trans. and Longi. PGD | Landslide |
|-------------------|------------------|----------|-----------------------|-----------|
| I                 | 1.5              | 2.3      | 1.5                   | 2.6       |
| II                | 1.3              | 1.5      | 1.4                   | 1.6       |
| III               | 1.0              | 1.0      | 1.0                   | 1.0       |

loads and permanent ground deformation (PGD) owing to seismic waves.

2. Theories on Seismic Waves

2.1 Criteria for Seismic Waves

The design of subsea pipelines must consider the resistance to the deformation of a ground surface owing to soil deformations in the fault zone. The size of an earthquake is expressed by its magnitude. Table 2.1 lists the seismic hazards in earthquake design, and Table 2.2 lists the peak ground acceleration (PGA) of the bedrock in each seismic zone.

2.2 Seismic Risk Analysis

Seismic risk analysis must be conducted for the excavation or burial of subsea pipelines. The major seismic risk factors of buried pipelines are as follows:
1) Strain, bending, and axial stress on the subsea pipeline by variations of waves in different locations along the pipelines.
2) Large strains from the movement of fault zones when an earthquake occurs through the subsea pipeline passes onto the fault zone.
3) Buoyancy of the pipeline and ground movement owing to the liquefaction of soil by an earthquake.

With regard to the risk of an earthquake in a subsea pipeline, the movements of fault zones or soil are small owing to the general flatness of the subsea ground and the deep fault zone. Furthermore, the progress of shear waves results from the axial stress that is parallel to the pipe axis direction, owing to the bending stress caused by the strain and deformation of the ground.

Table 2.1 Recommended design levels for seismic hazards

| Pipe class | Probability of exceeding in 50 years | Return period (years) |
|------------|-------------------------------------|-----------------------|
| I          | 2%                                  | 2475                  |
| II         | 5%                                  | 975                   |
| III        | 10%                                 | 475                   |
| IV         | No seismic design consideration required |
Table 2.2 Peak ground acceleration per seismic zone

| Seismic Zone | II  | III | IV  | V   |
|--------------|-----|-----|-----|-----|
| PGAr         | 0.1 | 0.16| 0.24| 0.36|

3. Estimation of Seismic Response Coefficient

This section examines the effects of seismic waves on the ground by considering the characteristics of seismic waves and the ground. The natural period of the ground surface is expressed by the following equation:

\[
T_G = c \times \sum \frac{H_i}{V_{si}}
\]  

(2.3)

where \( c = 4.0 \) (clay soil) or 5.23 (sandy soil), as per soil classification for cohesive effects \( H_i = i \)th soil layer thickness (m), and \( V_{si} = i \)th shear wave velocity (m/s).

Tables 3.1 and Table 3.2 list the ground amplification factors according to the shear wave velocity and acceleration by the characteristics of the soil.

Table 3.1 Ground amplification factor (Ig) for peak ground velocity for various soils

| Class of Soil | PGV/PGVr | PGVr ≤ 0.1 m/s | PGVr = 0.2 m/s | PGVr = 0.3 m/s | PGVr = 0.4 m/s | PGVr ≥ 0.5 m/s |
|---------------|----------|----------------|----------------|----------------|----------------|----------------|
| A             | 0.8      | 0.8            | 0.8            | 0.8            | 0.8            | 0.8            |
| B             | 1        | 1              | 1              | 1              | 1              | 1              |
| C             | 1.7      | 1.6            | 1.5            | 1.4            | 1.3            | 1.3            |
| D             | 2.4      | 2              | 1.8            | 1.6            | 1.5            | 1.5            |
| E             | 3.5      | 3.2            | 2.8            | 2.4            | 2.4            | 2.4            |

Table 3.1 Ground amplification factor (Ig) for peak ground acceleration for various soils

| Class of Soil | PGA / PGAr | PGAr ≤ 0.1 g | PGAr = 0.2 g | PGAr = 0.3 g | PGAr = 0.4 g | PGAr ≥ 0.5 g |
|---------------|------------|--------------|--------------|--------------|--------------|--------------|
| A             | 0.8        | 0.8          | 0.8          | 0.8          | 0.8          | 0.8          |
| B             | 1          | 1            | 1            | 1            | 1            | 1            |
| C             | 1.2        | 1.2          | 1.1          | 1            | 1            | 1            |
| D             | 1.6        | 1.4          | 1.2          | 1.1          | 1            | 1            |
| E             | 2.5        | 1.7          | 1.2          | 0.9          | 0.9          | 0.9          |

In the above tables, A: hard rock, B: rock, C: very dense soil and soft rock, D: dense/stiff soil, E1: loose/soft soil, E2: soft soil with PI > 10 and natural moisture content ≥ 40%, and F: peak or highly organic clays or very-high-plasticity clays. PGV and PGA are ground surface values and PGVr, PGAr are base rock values.

The soil damping coefficient during an earthquake is very important, and a damping ratio of 5–10% must be applied to the soil. Table 3.3 lists the damping coefficients according to damping ratios.

Table 3.3 Soil Damping Ratio and Coefficient

| Damping Ratio (%) | Damping Reduction | Damping Ratio (%) | Damping Reduction |
|-------------------|-------------------|-------------------|-------------------|
| 0.5               | 1.88              | 5                 | 1                 |
1.62 7 0.87
1.35 10 0.73
1.2 20 0.46

Furthermore, the risk level factors (I) considering the seismic return period according to the characteristics of structures are listed in Table 3.4.

Table 3.4 Seismic Risk Level and Probability

| Seismic Return Period, T_R | 45Y | 95Y | 224Y | 475Y | 975Y | 2475Y |
|---------------------------|-----|-----|------|------|------|-------|
| Risk Level Factor, I      | 0.44| 0.72| 1    | 1.25 | 1.5  | 2     |
| Probability of exceedance, P_R | 20% | 10% | 20%  | 10%  | 5%   | 2%    |
| Time span, T_L            | 10Y | 10Y | 50Y  | 50Y  | 50Y  | 50Y   |
| Probability of PQ, in Q (30) years | 49% | 27% | 13%  | 6%   | 3%   | 1%    |

Among the characteristics of the ground surface, the soil amplification factor varies by the characteristics of the ground surface. However, S is determined by Tables 3.1 and 3.2, and the shear wave velocity of an earthquake on the surface can be expressed using the following equation:

\[
V_s' = \sum \frac{H_i}{\sum V_{sl}} = C_s
\]  

(3.1)

Furthermore, the PGA used in the actual design can be expressed by the following equation by considering the risk level factor for the value found from the seismic response spectrum:

\[
PGA = PGA \times I
\]  

(3.2)

Fig. 3.1 shows the response spectrum acceleration of the earthquake: \( C_s (g) = 1.2AS/T^{2/3} \leq 2.5A \).

The wavelength of the seismic motion by earthquake is expressed by the following equation:

\[
L = \frac{2L_1 \times L_2}{L_1 + L_2}
\]  

(3.3)

where \( L_1 = T_G \times V_s' \), \( L_2 = T_G \times V_{BS} \), \( V_s' = \frac{\sum H_i}{\sum (H_i/\nu_i)} \), \( V_{BS} \) = shear seismic wave velocity of the ground base, and \( V_s' \) = average shear seismic wave velocity of the ground.

![Fig 3.1 Ground spectral acceleration](image)
If onland PGV and PGA are suggested by Seismic Classification report, the subsea pipeline seismic PGV, PGA shall be reduction based on depth of Soil Layer. Normally, the reduction is around 40–50% of Seismic Velocity and Acceleration onland ground surface.

4. Estimation of Stiffness of Soil and Stress by Seismic Wave

To examine the effects of earthquakes on subsea pipelines, the stiffness of the transverse, horizontal, and vertical soils can be calculated by Eqs. (4.1), (4.2), (4.3.1), (4.3.2), and (4.4), as recommended by ALA (2001).

Transverse stiffness, \( t_u = \pi D c \alpha + \pi D H \gamma '[(1 + k_0)/2] \tan \delta (kN/m) \) (4.1)

where the adhesion coefficient is \( \alpha = 0.608 - 0.123c - 0.274/c^2 + 0.695/c^3 + 1 \), the adhesion coefficient of the backfill soil is \( \alpha = c/100 \), \( H \) is the height from the center of the pipeline to the top of the backfill, and the internal friction angle of the ground and pipeline is \( \delta = f \varphi \).

| Pipe Coating  | \( f \) |
|---------------|---------|
| Concrete      | 1       |
| Coal Tar      | 0.9     |
| Rough Steel   | 0.8     |
| Smooth Steel  | 0.7     |
| FEB           | 0.6     |
| PE            | 0.6     |

### Table 4.1 Internal friction coefficient

| Type of soil     | \( \Delta t \) (mm) |
|------------------|---------------------|
| Dense sand       | 3                   |
| Loose sand       | 5                   |
| Stiff clay       | 8                   |
| Soft clay        | 10                  |

### Table 4.2 Maximum axial strain

### Table 4.3 Lateral bearing capacity factor of soil (ALA, 2001)

| Factor  | \( \varphi \) | \( a \) | \( b \) | \( c \) | \( d \) | \( e \) |
|---------|----------------|--------|--------|--------|--------|--------|
| \( N_{ch} \) | 0     | 6.752  | 0.065  | -11.063| 7.119  |
| \( N_{ch} \) | 20    | 2.399  | 0.439  | -0.03  | 1.059E-03| -1.754E-05|
| \( N_{ch} \) | 25    | 3.332  | 0.839  | -0.09  | 5.606E-03| -1.319E-04|
| \( N_{ch} \) | 30    | 4.565  | 1.234  | -0.089 | 4.275E-03| -9.159E-05|
| \( N_{ch} \) | 35    | 6.816  | 2.019  | -0.146 | 7.651E-03| -1.683E-04|
| \( N_{ch} \) | 40    | 10.959 | 1.783  | 0.045  | -5.425E-03| -1.153E-04|
| \( N_{ch} \) | 45    | 17.658 | 3.309  | 0.048  | -6.443E-03| -1.299E-04|

Lateral stiffness, \( P_u = N_{ch} S_u D + N_{qh} H D (kN/m) \) (4.2)

where \( N_{ch} = a + bx + c/(x + 1)^2 + d/(x + 1)^3 \leq 9, \ N_{qh} = a + bx + cx^2 + dx^3 + ex^4, \) and \( x = H/D \). Furthermore, \( \Delta_p = 0.04(H + 0.5D) \leq 0.01D \) and \( \Delta_p = 0.02D \) are used for the strain.

| \( \varphi \)  | \( a \) | \( b \) | \( c \) | \( d \) | \( e \) |
|---------------|--------|--------|--------|--------|--------|
| \( N_{ch} \)  | 0      | 6.752  | 0.065  | -11.063| 7.119  |
| \( N_{ch} \)  | 20     | 2.399  | 0.439  | -0.03  | 1.059E-03| -1.754E-05|
| \( N_{ch} \)  | 25     | 3.332  | 0.839  | -0.09  | 5.606E-03| -1.319E-04|
| \( N_{ch} \)  | 30     | 4.565  | 1.234  | -0.089 | 4.275E-03| -9.159E-05|
| \( N_{ch} \)  | 35     | 6.816  | 2.019  | -0.146 | 7.651E-03| -1.683E-04|
| \( N_{ch} \)  | 40     | 10.959 | 1.783  | 0.045  | -5.425E-03| -1.153E-04|
| \( N_{ch} \)  | 45     | 17.658 | 3.309  | 0.048  | -6.443E-03| -1.299E-04|

Upward stiffness, \( Q_u = N_{cv} S_u D + N_{qv} H D (kN/m) \) (4.3.1)

where \( N_{cv} = 2(H/D) \leq 10, \ H/D \leq 10, \ N_{qv} = \varphi H/(44D) \leq N_q, \) and \( N_q = e^{\pi \tan \varphi \tan 2(45 + \varphi/2)} \).

Downward stiffness, \( Q_d = N_c S_u D + N_q H D + N_{y Sat} D^2/2 (kN/m) \) (4.3.2)

where \( N_c = [\cot(\varphi + 0.001)] \left[e^{\pi \tan(\varphi + 0.001)} \tan^2(45 + \varphi/2) - 1 \right] - 1 \)
\[ N_q e^{\pi \tan \phi \tan^2 (45 + \phi/2)}, N_p = e^{0.18\phi - 2.5} \]

| Type of soil  | \( \Delta q_u \) | Remark |
|--------------|-----------------|--------|
| Dense sand   | 0.01H <0.1D     |        |
| Loose sand   | 0.02H <0.1D     |        |
| Stiff clay   | 0.1H <0.2D      |        |
| Soft clay    | 0.2H <0.2D      |        |

### 5. Stress and Strain on Subsea Pipeline

Before reviewing the deformation by earthquake, the longitudinal deformation caused by the pressure and temperature difference owing to the internal pressure and the earthquake is expressed by the following equation recommended by Ramberg Osgood’s stress strain relationship (1943):

\[ \varepsilon = \frac{\sigma}{E_t} \left[ 1 + \frac{n}{1+r} \left( \frac{\sigma}{\sigma_y} \right)^r \right] \quad (5.1) \]

where the stress of the subsea pipeline owing to the pressure increase caused by internal pressure and the earthquake is

\[ \sigma = \frac{(P_i + P_d)\mu D}{2t} \quad and \quad \sigma = E_t \alpha (T_{2d} - T_3), \quad \mu = \text{Poisson's ratio}, \]

\[ P_i = \text{internal pressure, and } P_d = \text{seismic pressure}. \]

Furthermore, with regard to the deformation of subsea pipelines by earthquakes, length \( L \) and width \( W \) are calculated for the PGD considering the elastoplastic stiffness of the soil in the axial and transverse directions of the subsea pipeline, in order to examine the deformation of subsea pipelines by the deformation type of the soil. In addition, Figs. 5.1 and 5.2 are examined.

The strain on the subsea pipeline according to the longitudinal PGD for landsliding can be defined by the distance of the PGD in Eq. (5.2) and the ground deformation of a long distance by PGD in Eq. (5.3), which can be expressed as \( \delta_{\text{design}} \). This was recommended by GSDMA (2007), which was modified from O’Rourke et al. (1995).

\[ \varepsilon_a = \left[ \frac{(t_u L)}{(2\pi D \ t_e)} \right] \left[ 1 + n/(1+r) \right] \left[ t_u L/(2\pi D \ t_e \sigma_y) \right]^r \quad (5.2) \]

\[ \varepsilon_a = \left[ \frac{(t_u L)}{(2\pi D \ t_e)} \right] \left[ 1 + n/(1+r) \right] \left[ t_u L/(2\pi D \ t_e \sigma_y) \right]^r \quad (5.3) \]

Here, for the deformation of the longitudinal length, the minimum values for Eqs. (5.2) and (5.3) are used. Furthermore, \( \delta^1 = L \varepsilon_a \) and \( \delta_{\text{design}}^1 = \delta^1 L_p \).

![Fig. 5.1 Parallel to the ground movement](image1)

![Fig. 5.2 Transverse to the ground movement](image2)
Table 5.1 Ramberg-Osgood parameters for steel pipes

| Grade of Pipe | Grade B | X-42 | X-52 | X-60 | X-70 | Ductile iron |
|---------------|---------|------|------|------|------|--------------|
| n             | 10      | 15   | 9    | 10   | 5.5  | 15           |
| r             | 100     | 32   | 10   | 12   | 16.6 | 32           |

\[
\delta_{\text{design}} = \left[ \frac{1}{\pi} \ln \left( \frac{2\sigma_f}{E} \right) \right] \left( 1 + \frac{2}{2+n} \right) \left( \frac{n}{1+n} \right) \left( t_u L_e / (2\pi D t e) \right)^r
\]

Here, \( n \) and \( r \) are the Ramberg-Osgood parameters from Table 5.1.

Furthermore, the longitudinal PGD (cm) of the soil \( \delta \) can be expressed as the following equation recommended by Jibson & Keefer (1993):

\[
\log(\text{PGD}) = 1.546 + 1.460 \log (I_A) - 6.642 a_c \quad (5.4)
\]

where the Aruaus intensity (m/s) is \( I_A = -4.1 + M_w - 2 \log(R) \) (Wilson & Keefer 1985), \( R \) is the distance from the center of the earthquake, the critical acceleration of soil (g) is \( a_c = g (FS - 1) \sin(\alpha) \) as recommended by Newmark(1965), \( FS = (S_u + H \gamma' \cos(\alpha) \tan(\varphi)) / (H \gamma' \sin(\alpha)) \), and \( \alpha \) and \( \varphi \) are the tilt angle and internal friction angle of the soil, respectively.

Furthermore, to determine the longitudinal length \( L \), we can use FE analysis considering the rigid block pattern of landsliding (as shown in Fig. 5.3), the buried length \( L_{\text{em}} \) of Eq. (5.5) suggested by Flores-Berrones and M. O’Rourke (1992), or the longitudinal peak length displacement of PGD of Eq. (5.6). The deformation of the pipeline according to the longitudinal length \( L \) can be expressed as Eq. (5.7).

\[
L_{\text{em}} = \sqrt{\frac{PGD \cdot AE}{t_u}} \leq \frac{L}{2} \quad (5.5)
\]

Peak pipe deformation, \( \varepsilon = \sqrt{\frac{t_u L \cdot PGD}{EA}} \leq \frac{t_u L}{2EA} \quad (5.6)
\]

\[
\varepsilon = \frac{\alpha L}{2L_{\text{em}}} \text{, when } L < 4L_{\text{em}}, \quad \varepsilon = \alpha \sqrt{\frac{L}{L_{\text{em}}}} \text{, when } L \geq 4L_{\text{em}} \quad (5.7)
\]

If Eq. (5.6) is equal to Eq. (5.7), the longitudinal length \( L \) must be determined by the Newton-Raphson method, where the ground deformation is \( \delta = \frac{L}{L} \) and \( \delta = \frac{PGD}{L} \).

If the PGD is a ridge pattern, as shown in Fig. 5.4, the buried length \( L_{\text{em}} \) can be expressed as Eq. (5.8), and the longitudinal peak length deformation of PGD can be expressed as Eq. (5.9). The pipe deformation according to the longitudinal length \( L \) can be expressed as Eq. (5.10).

\[
L_{\text{em}} = \sqrt{\frac{\alpha^2 + \frac{t_u L \cdot \alpha}{EA}}{\alpha^2 + \frac{t_u L \cdot \alpha}{EA} - \alpha}} \quad (5.8)
\]

Peak pipe deformation, \( \varepsilon = \frac{t_u L_{\text{em}}}{EA} = \sqrt{\frac{\alpha^2 + \frac{t_u L \cdot \alpha}{EA}}{\alpha^2 + \frac{t_u L \cdot \alpha}{EA} - \alpha}} \quad (5.9)
\]

\[
\varepsilon = \alpha \sqrt{1 + \frac{L}{L_{\text{em}}} - 1}, \text{ when } L < 3L_{\text{em}}, \quad \varepsilon = \alpha, \text{ when } L \geq 3L_{\text{em}} \quad (5.10)
\]

If Eq. (5.9) is equal to Eq. (5.10), the longitudinal length \( L \) must be determined by the Newton-Raphson method, where the ground deformation is \( \alpha = \frac{2\delta}{L} \) and \( \delta = \frac{PGD}{L} \).
Furthermore, if the PGD is equal to Fig. 5.5, which is a ramp pattern, the longitudinal buried length $L_{em}$ is expressed as Eq. (5.11), and the peak pipe deformation is expressed as Eq. (5.12), which can be expressed as Eq. (5.13) by using the buried length $L_{em}$.

$$L_{em} = \frac{PGD \cdot AE}{\tau_u} \quad (5.11)$$

$$\varepsilon = \sqrt{\frac{\alpha L \cdot \tau_u}{EA}} \leq \alpha \text{ or } \frac{\tau_u L_{em}}{EA} \quad (5.12)$$

$$\varepsilon = \alpha \sqrt{\frac{L}{L_{em}}} \text{ when } L < L_{em} \quad \varepsilon = \alpha \text{ when } L \geq L_{em} \quad (5.13)$$

In the above equations, the ground deformation is $\alpha = \frac{\delta}{L}$, and $\delta = PGD$. The longitudinal length $L$ can be determined by assuming that Eq. (5.12) is identical to Eq. (5.13).

In addition, the PGD is equal to the ramp/step pattern in Fig. 5.6. If the pipe deformation $\varepsilon$ is smaller than the ground deformation $\alpha$, the longitudinal buried length $L_{em}$ can be expressed as Eq. (5.14), and the maximum pipe deformation can be expressed by Eq. (5.15). Using the buried length $L_{em}$, the pipe deformation can be expressed as Eq. (5.16).

$$L_{em} = \sqrt{\frac{4\alpha^2 + 2\tau_u \alpha L/EA - 2\alpha}{\tau_u/EA}} \quad (5.14)$$

$$\varepsilon = \sqrt{4\alpha^2 + 2\tau_u \alpha L/EA - 2\alpha} \text{ or } \frac{\tau_u L_{em}}{EA} \quad (5.15)$$

$$\varepsilon = \alpha \left[\sqrt{4 + \frac{2L}{L_{em}} - 2}\right] \text{ When } L < 2.5L_{em} \quad \varepsilon = \frac{\alpha}{2} \left[\sqrt{\frac{4L}{L_{em}} - 1} - 1\right] \text{ When } L \geq L_{em} \quad (5.16)$$

If the pipe deformation $\varepsilon$ is equal to the ground deformation $\alpha$, the buried length $L_{em}$ can be expressed by Eq. (5.14), and the pipe deformation by the buried length $L_{em}$ can be expressed by Eq. (5.16). Furthermore, the longitudinal lengths $L$, $L_1$, $L_2$, and $L_3$ can be expressed by Eq. (5.17), and the peak pipe extrusion deformation can be expressed by Eq. (5.18).

$$L_1 = \frac{\alpha EA}{\tau_u}, \quad L_2 = \frac{\tau_u}{\alpha EA} \left( L_3^2 - L_2^2 \right), \quad L_3 = \sqrt{\frac{4\tau_u L - 1}{\alpha EA}}, \quad L = 1.5L_1 + L_2 + L_3 \quad (5.17)$$

$$\varepsilon = \frac{\tau_u L_3}{EA} = \frac{\alpha}{2} \left[\sqrt{\frac{4\tau_u L}{\alpha EA} - 1} - 1\right] \quad (5.18)$$

Here, the ground deformation is $\alpha = \frac{\delta}{L}$, and $\delta = PGD$. If $\varepsilon < \alpha$, the longitudinal length $L$ can be calculated by assuming that Eq. (5.15) is equal to Eq. (5.16). If $\varepsilon = \alpha$, the longitudinal length $L$ can be calculated by assuming that Eq. (5.18) is equal to Eq. (5.16).

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**Fig. 5.3 Rigid block pattern**

**Fig. 5.4 Ridge pattern**
To examine the subsea pipeline deformation for the longitudinal PGD, Eqs. (5.19) and (5.20), which were recommended by O’Rourke et al. (1999), represent the lateral broad width (W) and lateral narrow width (W), respectively. The smaller value between them must be applied.

\[
\varepsilon_b = \pm \pi D \delta_{\text{design}}^2 / W^2 \quad (5.19)
\]

\[
\varepsilon_b = \pm (P_u W^2) / (3\pi E t D^2) \quad (5.20)
\]

Here, \( P_u \) is the lateral resistance (kN/m) for the ground condition, and \( W \) is the lateral width of the PGD. In addition, the lateral width \( \delta^l \) is expressed as Eq. (5.4), and \( W \) can be expressed as Eqs. (5.21) and (5.22), which were recommended by O’Rourke (1989) for narrow landsliding and broad landsliding, respectively.

\[
W_b = \sqrt{\frac{3.84EIP_{\text{PGD}}}{5P_u}} \quad \text{for 2-end hinged beam deformation} \quad (5.21)
\]

\[
W_b = W_b + W_a, W_a = \frac{\pi^2P_{\text{GD}}^2}{4W_a} = \frac{\sigma W_a}{E} + \frac{\theta^2}{E t_u}, \sigma = \frac{P_u W_a^2}{16A P_{\text{GD}}} \quad (5.22)
\]

Furthermore, the lateral PGD caused by liquefaction can be expressed as Eq. (5.23) as suggested by Hamada et al. (1986), the slope inclination by Eq. (5.24) as suggested by Bartlett and Youd (1992), and the free breaking envelope by Eq. (5.25), as follows:

\[
PGD = 0.75\sqrt{R^3/\theta} \quad (m) \quad (5.23)
\]

\[
\log(PGD + 0.01) = -15.787 + 1.178M_w - 0.927\log(R^*) - 0.013R^* + 0.429\log(S) + 0.348\log(T_{15}) + 4.527\log(100 - FC_{15}) - 0.922D_{50_{15}} \quad (5.24)
\]

\[
\log(PGD + 0.01) = -15.787 + 1.178M_w - 0.927\log(R^*) - 0.013R^* + 0.429\log(Y) + 0.348\log(T_{15}) + 4.527\log(100 - FC_{15}) - 0.922D_{50_{15}} \quad (5.25)
\]

where \( M_w \) is the magnitude of the earthquake, \( R^* = R_0 + R = 10^{0.89M_w+5.64} \), \( R \) is the horizontal distance to the fault zone (km), \( T_{15} \) is the soil thickness (m) of SPT at N=15 or lower, \( FC_{15} \) is the silt sand content (%) of \( T_{15} \), \( D_{50} \) is the average particle diameter (mm) of \( T_{15} \), and \( S = Y = 100A/B \). In the case of the ground slope, \( A \) is the slope height, and \( B \) is the horizontal distance of the slope. Furthermore, for \( Y \), which is the ground free surface failure, \( A \) is the height of the slope inclination, and \( B \) is the horizontal distance of the breaking curve.

Therefore, the lateral width \( W \) and longitudinal length \( L \) caused by ground liquefaction can be expressed by Eq. (5.26) suggested by Suzuki and Masuda (1991), and Eq. (5.27) suggested by Bartlett and Youd (1992).

\[
W = PGD / 0.003 \quad (5.26)
\]

\[
L = 300 - 58.333PGD \quad (5.27)
\]
Also, $\delta_{\text{design}}^l = \delta^l I_p$, $I_p = \varepsilon \text{portance Factor}$.

Furthermore, the displacement owing to the upward load caused by the liquefaction of the ground by the earthquake in the subsea pipeline is expressed by Eq. (5.28) for the narrow liquefaction area of the upward length, which was recommended by GDSMA (2007) using the Ramberg-Osgood variable, and by Eq. (29) for the broad liquefaction area of the upward length, suggested by O’Rourke et al. (1999). The smaller of these two values is used.

\[
\varepsilon_1 = \frac{\sigma_{bf}}{E_s} \left[ 1 + \frac{n}{1+n} \left( \frac{\sigma_{bf}}{\sigma_y} \right)^m \right] \tag{5.28}
\]

\[
\varepsilon_2 = \frac{\pi^2 D \delta^F}{L_b^2} \pm \frac{\pi^2 \delta^F^2}{4L_b^2} \tag{5.29}
\]

Here $\sigma_{bf}$ and $\delta^F$ are as recommended by A LA (2005), $F_b = W_s - [W_p + W_w + (P_v - \gamma_w h_w)D]$ as recommended by A LA (2005), $P_v = \gamma_w h_w + R_w Y' C + 2c(C/D)$, and the buoyancy coefficient $R_w = 1 - 0.33(\frac{h_w}{C}) \leq 1.0$, $C = S_u (kPa)$, $C = \text{Backfill cover depth}$, $W_s = A_c Y_{sat}$, and $\delta^F$ is the floating height of pipe owing to liquefaction.

Furthermore, before $L_b$ can be determined, the actual upward PGD by liquefaction must be obtained, or the PGD for the downward subsidence must be determined.

The upward PGD can be expressed by the following equation, and the downward subsidence is ignored:

\[
\text{PGD} = \delta^F = \frac{F_b}{A Y_{sat}} \tag{5.30}
\]

where $A$ is the similar area of subsea pipeline ($m^2$), and $Y_{sat}$ is the saturated unit weight of soil (kN/m$^3$).

Furthermore, the liquefaction of the pipe and upward length $L_b$ can be expressed by the following Eqs. (5.31) and (5.32) for the narrow distance and for the broad length, which were obtained by using the equation recommended by O’Rourke (1989).

\[
L_b = \frac{A_{\text{EI}} \text{PGD}}{5P_u} f_{2 \text{ for 2 - end hinged beam deformation}} \tag{5.31}
\]

\[
L_b = L_{b'} + L_{a'} l_s \frac{\pi^2 \text{PGD}^2}{4L_a} = \frac{\sigma I_{a}}{E} + \frac{A \sigma^2}{E' I_{a}}, \sigma = \frac{P_u k_a^2}{16A \text{PGD}} \tag{5.32}
\]

Furthermore, the effect of the subsea pipeline when an earthquake occurs by the fault of the ground can be divided into the following: (1) forward moving stroke, (2) static fault zone, 3) inverse fault zone, and (4) general fault zone, as shown in Figs. 5.7 and 5.8.
Next, to consider the displacement of the fault zone by an earthquake, the recommended formulas by Newmark and Hall (1975) and Wells and Coppersmith (1994) can be expressed as Eqs. (5.33), (5.34), (5.35), and (5.36), respectively:

Main movement shear direction $\log \delta_{fr} = -6.32 + 0.90M$ (5.33)

Forward fault $\log \delta_{fn} = -4.45 + 0.63M$ (5.34)

Inverse short layer is $\log \delta_{fr} = -0.74 + 0.08M$ (5.35)

Weak fault zone or underground layer, $\log \delta_{fb} = -4.80 + 0.69M$ (5.36)

The components in the installation direction of the subsea pipeline are listed in Table 5.2.

| Main direction movement fault layer | Forward fault | Inverse fault | Peak design displacement |
|-------------------------------------|---------------|---------------|-------------------------|
| $\delta_{fax}$ = $\delta_{fax} \cos \beta$ | $\delta_{fax} \cos \psi \sin \beta$ | $\delta_{fax} = \delta_{fx} \cos \psi \sin \beta$ | $\delta_{fax-design}$ = $\delta_{faxI_p}$ |
| $\delta_{fr}$ = $\delta_{fr} \sin \beta$ | $\delta_{fr} \cos \psi \cos \beta$ | $\delta_{fr} = \delta_{fr} \cos \psi \cos \beta$ | $\delta_{fr-design}$ = $\delta_{frI_p}$ |

$\beta$ = angle of pipeline crossing a fault line, $\psi$ = Dip angle of the fault

Thus, the displacement by fault can be expressed by the following equation:

$$\varepsilon = 2 \left[ \left( \frac{\delta_{fax-design}}{2L_a} \right) + \frac{1}{2} \left( \frac{\delta_{fr-design}}{2L_a} \right) \right] + \frac{1}{2} \left( \frac{\delta_{fr-design}}{2L_a} \right) (5.37)$$

where $L_a = \frac{E_y \epsilon_y \pi D t}{Q_u} + \frac{E_p \epsilon_p \pi D t}{Q_u}$ (m),

$\varepsilon_y$ is the yield displacement, $E_i$ is the coefficient of elasticity, and $\epsilon_p$ and $E_p$ are the plastic deformation and coefficient, respectively.

Last, the deformation of the subsea pipeline by seismic waves can be expressed by Eq. (5.38), which was recommended by Newmark’s method (Yeh, 1974) and ALA guidelines (2001).
As shown in Figs. 5.9 and 5.10, the longitudinal length deformation of the pipeline by S-Wave and R-Wave is expressed by the following equation:

\[ \varepsilon_a = \frac{V_g}{C_{\text{apparent}}} = \frac{V_g}{\alpha_s C} \] (5.38)

where \( V_g = PGV \cdot l_p \) and \( C_{\text{apparent}} \) is divided into \( C_s_{\text{apparent}} \) and \( C_r_{\text{apparent}} \), \( C_s_{\text{apparent}} = C_s / (\sin \gamma \cos \gamma) \), and \( C_r_{\text{apparent}} = C_{rph} \).

In general, if the S-wave and R-wave phase velocities of the earthquake cannot be known, \( C \) is the seismic wave propagation velocity, which is approximately 2,000 m for the S wave and approximately 500 m for the R wave. Furthermore, \( \alpha_s \) is the ground deformation coefficient, which is 2.0 for the S wave and 1.0 for the R wave. If this is an S-wave, the peak value of \( \varphi = \gamma = 45^\circ \) is used; otherwise, the radius of curvature of the subsea pipeline owing to the S wave is \( k_g = PGA / C^2 \), and \( \varepsilon_a = k_g D / 2 \) must be used.

Furthermore, \( C_s = \Sigma H \cdot V_{SI} / \Sigma H \) must be used. O’Rourke et al. (1984) recommended \( C_{rph} = \lambda f \) and \( f = 1/T_g \), but their applicability is separately regulated according to the value of \( H_f / C_L \). Therefore, the following can be established:

\[ C_{rph} = 0.875 C_H \text{ when } H_s f / C_L \leq 0.25 \]
\[ = 0.875 C_H - \frac{0.875 C_H - C_L}{0.25} \left( \frac{H_s f}{C_L} - 0.25 \right), \text{ when } 0.25 < \frac{H_s f}{C_L} \leq 0.5 \]
\[ = C_L \text{ when } H_s f / C_L > 0.5 \]

where \( C_L = \Sigma H_i V_{SI} / \Sigma H_i \), \( H_i \) and \( V_{SI} \) are the shear velocity and depth on the soil above the bedrock, respectively, and the bedrock shear wave velocity is \( C_H = V_{SI} \).

Furthermore, the longitudinal peak deformation owing to the friction of the ground can be expressed by Eq. (5.39) according to the wavelength by the seismic wave length \( L (=\lambda) \), which is the equation of O’Rourke (2003) recommended by ALA guidelines (2001).

\[ \varepsilon_a = \frac{t_{\text{HL}} \lambda}{4AE_s} \] (5.39)

Thus, a smaller value between Eqs. (5.38) and (5.39) should be selected and used.

In addition, the internal pressure increases by the vibration of internal fluid by the earthquake. This increase in internal pressure must be examined. If the internal pressure influences the thickness of the steel pipe, the thickness of the steel pipe must be changed accordingly. The following equation determines the increase in internal pressure:

\[ P_d = \frac{\pi D^2}{4} K_h Y_c \] (5.40)

where \( K_h = PGA \), and \( Y_c = \) internal fluid unit weight. Thus, the dynamic fluid pressure for \( P_d \), the internal pressure \( P_i \) in Eq. (5.1) must be added to the operating pressure.

6. Conclusions

This study found that if the strain and stress of an earthquake are within the allowable strain and allowable deformation when subsea pipelines are designed, there will be no structural problems caused by the earthquake. Furthermore, the deformations during operation must be considered by dividing them into a tension part and a compression part.

Table 6.1 shows the allowable strains for steel pipes and other materials.
Table 6.1 Allowable Strains of Pipes (Recommended practice DNV-RP-C208)

| Continuous Oil and Gas Pipeline | Allowable Strain | Tension | Compression |
|---------------------------------|------------------|---------|-------------|
| Ductile Cast Iron Pipe          | 2%               |         | For PGD: Onset of Wrinkling ($\varepsilon_{cr}$) |
| Steel Pipe                      | 0.4%             |         |             |
| Polyethylene Pipe               | 20%              |         |             |
| Bends and Tees of pipe          | 1%               |         | Wave propagation: 50% to 100% of the Onset of Wrinkling ($0.5$ to $1 \varepsilon_{cr}$) |

| Continuous Water Pipeline       | Allowable Strain | Tension | Compression |
|---------------------------------|------------------|---------|-------------|
| Steel water                     | 5%               | $\varepsilon_{cr-pgd}$ |             |
| Iron pipe water                 | 5%               | $\varepsilon_{cr-wave}$ |             |

In the above table, $\varepsilon_t = \text{tensile strain of pipe}$, $\varepsilon_p = \text{tensile failure strain of pipe}$, $\varepsilon_{cr-pgd} = 0.88 \frac{f}{R}$, $\varepsilon_{cr-wave} = 0.75 \left[ \frac{0.5f}{D'} - 0.0025 + 3000 \left( \frac{PD}{2E_t} \right)^2 \right]$, $D' = \frac{D}{1 - 0.0025D}$ ($D - D_{\text{min}}$), $D_{\text{min}} = D$ - tolerance of wall thickness, $\varepsilon_{cr-c} = 0.175 \frac{f}{R}$, and the theoretical local compression buckling is compression strain $\varepsilon_c = 0.6 \frac{f}{R}$.

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