Lab-scale impact test to investigate the pipe-soil interaction and comparative study to evaluate structural responses

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ABSTRACT: This study examined the dynamic response of a subsea pipeline under an impact load to determine the effect of the seabed soil. A laboratory-scale soil-based pipeline impact test was carried out to investigate the pipeline deformation/strain as well as the interaction with the soil-pipeline. In addition, an impact test was simulated using the finite element technique, and the calculated strain was compared with the experimental results. During the simulation, the pipeline was described based on an elasto-plastic analysis, and the soil was modeled using the Mohr-Coulomb failure criterion. The results obtained were compared with ASME D31.8, and the differences between the analysis results and the rules were specifically investigated. Modified ASME formulae were proposed to calculate the precise structural behavior of a subsea pipeline under an impact load when considering sand- and clay-based seabed soils.

KEYWORDS: Subsea pipeline; Pipe impact test; Pipeline-soil interaction; Pipe deformation; Finite element analysis.

INTRODUCTION

For the development of oil and gas reserves, a number of subsea pipelines have been installed and are under operation in the oceans around the world, such as in continental shelves and deep water regions. In addition, the installation and operation of subsea pipelines in deep ocean areas are more complicated than in shallow water. The conditions in deep water are extremely harsh. For example, a subsea pipeline should resist approximately 300-400 bar of hydraulic pressure and temperatures of 250-300 °C.

Approximately 50% of the various subsea equipment/facilities around the world have experienced failures in their subsea pipelines in recent decades. A failure accident can be categorized according to its mechanical/chemical aspects: corrosion, fatigue, and impact failures, which are induced by sea water; cyclic loads, such as from currents; and falling objects, such as subsea piles, respectively. Many studies on failure accidents have been carried out. Among them, corrosion and fatigue failures have been widely studied. In particular, many classification society rules have been put in place, and many research papers have been written.

Xue (2006) examined the problem of buckle propagation in corroded subsea pipelines, the variation in propagation pressure...
according to the form of the pipe, and the collapse mode in a corroded pipeline. The finite element model was validated using Timoshenko’s classical solutions. Pinheiro and Pasqualino (2009) proposed a methodology for a fatigue analysis of a damaged steel pipeline under cyclic internal pressure. This methodology uses the stress concentration factors, which are commonly used to modify the standard S-N curves of metallic structures under a high cycle fatigue load. The authors conducted fatigue tests to evaluate the finite life behavior of small-scale damaged pipes under cyclic internal pressure. The results of a fatigue analysis coincided well with the fatigue test results. Bazan and Beck (2013) proposed a non-linear model, in which the corrosion rate is represented as a Poisson square wave process. The resulting model represents the inherent time-variability of corrosion growth and was adjusted to the same set of actual corrosion data for two inspections from existing corrosion models. The proposed non-linear random process corrosion growth model led to the best fit to date, and more properly represents the physics of this problem.

Moreover, the design criteria for the corrosion and fatigue, among other factors, are specified in rules such as ABS (2008), API RP 1111 (2009), API 5L (2008), ASME B31.8 (2010), CSA Z662-07 (2007), DNV OS F101 (2010), and ISO 13623 (2009).

Nevertheless, although a number of codes and research papers related to two types of failure problems have been published, there have been few reports on impact failures. Jones et al. (1992) carried out a series of impact tests for a mild steel pipe by dropping a wedge-shaped object. In particular, the relationships between the kinetic energy and minimum internal and external diameters were investigated. Yang et al. (2009) conducted a pipe-on-pipe impact test to investigate the dynamic response of a pipe for the pipe impact problem. In their study, the effects of the impact location and pipe wall thickness were identified. Zeinoddini et al. (2013) evaluated the effect of soil-pipe interaction on the structural response during a pipe impact event using finite element calculations. A modified-Drucker-Prager/Cap plasticity model was adopted to describe the seabed soil, and an FEA technique considering the soil-pipe interaction was proposed and validated through a comparison with the reported test results.

ASME B31.8 (2010) provides calculation formulae for the amount of deformation and strain of a subsea pipeline under an impact load. In this code, the strain on the internal and external pipe surfaces is evaluated based on the bending strain in both the circumferential and longitudinal directions, and on the extensional strain in the longitudinal direction.

The aforementioned research papers and code focused on the pipe impact phenomenon and its associated structural behavior, but they have a significant limitation in that the foundation beneath the pipeline was postulated as being rigid. In addition, in some studies, the soil scale effect was not investigated during the experiments and/or simulations. In order to analyse the structural behavior of the subsea pipeline precisely, it is strongly recommended that the effect of interaction between pipe structures and sea-bed (soil) should be considered in the numerical analysis (Yu et al., 2013).

Therefore, in the present study, to assume the pipe behaviors in a subsea environment, both the precise pipe behaviors and the failure characteristics were investigated by considering the soil effect, which is difficult to predict and realize. In other words, a pipe impact test was carried out using a soil chamber and dry drop impact mass. To consider the various types of seabed soil, three types of soil, i.e., clay with 60% and 80% moisture content and water-saturated sand, were adopted for the experiments. During the test, a large scale soil chamber was used to avoid an abnormal reaction force generated by the narrow boundary conditions of the soil chamber.

In the pipe impact simulation, the soil was described based on the Mohr-Coulomb failure criterion. In addition, the soil scale effect was identified through a parametric study with respect to the seabed soil length and breadth. The analysis results were compared with the test results to validate the proposed simulation technique.

A series of numerical analyses were also conducted, and the results were compared with one of the existing subsea pipeline-related classification society rules, e.g., ASME B31.8 (2010). The differences between the FEA results and rule calculation results were investigated, and modified ASME formulae were proposed to calculate the precise structural behavior of a subsea pipeline under an impact load when considering sand- and clay-based seabed soils.

LABORATORY SCALE IMPACT TEST

A parametric laboratory-scale impact test was conducted to simulate the failure phenomenon of a subsea pipeline. The laboratory-scale impact test used in this study, which is the first attempt to simulate the impact response of a pipe system under a subsea environment, requires a combination of two test methods. The first method is an impact test method use to evaluate the
response of the target structure subjected to an impact load. The second method is a soil test method used to identify the soil characteristics (Vesic, 1971; Terzaghi et al., 1996; Das, 1998; Poulos, 2005). It was confirmed that the combination of the appropriate impact test and identification of soil interaction effect can be used as a systematic approach to estimate the impact behavior of a pipe system laid on top of a seabed.

**Experimental apparatus**

When a subsea pipeline system is installed on top of a seabed, there is a possibility that collisions will occur on the pipeline from the falling objects, such as ship anchors, during operation. Therefore, it is imperative to evaluate the performance of a subsea pipeline system with respect to the impact load from a falling object. In this context, an impact test facility was constructed to carry out dry drop tests using a free falling mass and height on a subsea pipeline system, as shown in Fig. 1. The drop height and weight can be readily controlled, and impact sensors are instrumented at the bottom of the impact test apparatus to measure the impact reaction forces. The impact force sensors type used for the experiment is a Model 200C50 S/N (4553, 4554) ICP. In addition, the strain is measured using a strain gauge attached to the pipe, as shown in Fig. 2. The strain gauge may be exposed to a humid environment because an impact test is conducted on watery sand (or clay). Moreover, a detachment of the strain gauge may occur during an impact incident. Therefore, to protect the strain gauge from a humid environment and an impact incident, it was attached near the collision area of the pipe. The strain along the longitudinal direction of the pipe was measured using a no. 1-3 strain gauge. The strain gauge type used for the experiment is an FLA 3-11 Lot No (A51551A). In addition, a deformation of the pipe was observed through high-speed photography. The camera used was Model IDT Y5. This high-speed camera offers high definition with a 4 megapixel sensor capable of 68,000 fps, and can therefore capture the deformation of a pipe in detail, along with the impact event between the dropping object and pipe.

![Fig. 1 Schematic diagram of the impact experiment.](image1)

![Fig. 2 Photograph of the strain gauge attachment location.](image2)
Investigation of the soil characteristics and identification of the foundation

Consideration of the seabed conditions: Soil foundation

There are many different types of soil in a seabed, such as sand, clay, and silt. In this study, two kinds of soil, i.e., sand and clay, were used as the foundation for a pipeline. Therefore, standard Jumoonjin sand was used as the test sand, as shown in Fig. 3(a), the properties of which are listed in Table 1. Given the water content of sandy soil, its relative density was 35%. Kaolinite was used to simulate a clay soil, as shown in Fig. 3(b). In addition, because kaolinite presents the different shear strength according to the moisture content, which was 60% and 80% in this experiment, its properties were examined and are listed in Table 2.

![Fig. 3 Experimental standard (a) sand and (b) clay.](image)

Material properties of the soil

The material property tests described below were conducted to identify the physical properties of the sand and clay used in the impact test. First, a specific gravity test was performed to determine the specific gravity of the materials (ASTM D854-14, 2006). A sieve analysis test was applied to determine the effective size, as well as and the coefficient of uniformity and curvature, using the size distribution curve shown in Fig. 4 (ASTM D136-06, 2005). As a result, the experimental material was sand with a uniform grain size. The internal friction angle was measured using a direct shear test (ASTM D3080, 2004). A compaction test was conducted to determine the maximum dry density ($\gamma_{d,max}$) (ASTM D4253-00, 2006). The minimum dry density ($\gamma_{d,min}$) was found by applying the ASTM test regulations (ASTM D4254-00, 2006). The shear strength was determined by conducting a vane shear test, as shown in Fig. 5 (ASTM D2573-08, 2001). The vane shear test applied a relatively rapid and economical in-situ method for determining the peak remolded undrained shear strength of very soft to medium-stiff clay. The test involved pushing a four-bladed vane into the clay stratum and rotating it slowly while measuring the resisting torque. In addition, a liquid limit test was conducted to determine the liquid limit (ASTM D4318, 2010). Through these tests, the values listed in Tables 1 and 2 were determined.

![Fig. 4 Grain-size distribution curves.](image)
Table 1 Material properties of the sand.

<table>
<thead>
<tr>
<th>No.</th>
<th>Title</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Specific gravity ( (G_s) )</td>
<td>-</td>
<td>2.63</td>
</tr>
<tr>
<td>2</td>
<td>Internal friction angle ( (\Phi) )</td>
<td>( ^\circ )</td>
<td>30.0</td>
</tr>
<tr>
<td>3</td>
<td>Dry unit weight ( (\varrho_d) )</td>
<td>( kN/m^3 )</td>
<td>14.74</td>
</tr>
<tr>
<td>4</td>
<td>Maximum dry unit weight ( (\varrho_{d_{\text{max}}}) )</td>
<td>( kN/m^3 )</td>
<td>16.70</td>
</tr>
<tr>
<td>5</td>
<td>Minimum dry unit weight ( (\varrho_{d_{\text{min}}}) )</td>
<td>( kN/m^3 )</td>
<td>13.80</td>
</tr>
<tr>
<td>6</td>
<td>Effective size ( (D_{10}) )</td>
<td>( mm )</td>
<td>0.32</td>
</tr>
<tr>
<td>7</td>
<td>Uniformity coefficient ( (C_u) )</td>
<td>-</td>
<td>1.90</td>
</tr>
<tr>
<td>8</td>
<td>Coefficient of gradation ( (C_c) )</td>
<td>-</td>
<td>1.00</td>
</tr>
<tr>
<td>9</td>
<td>Unified soil classification system (USCS)</td>
<td>-</td>
<td>SP</td>
</tr>
</tbody>
</table>

Fig. 5 Vane shear test equipment.

Table 2 Material properties of the clay.

<table>
<thead>
<tr>
<th>No.</th>
<th>Title</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Moisture content ( (w_w) )</td>
<td>%</td>
<td>60 80</td>
</tr>
<tr>
<td>2</td>
<td>Cohesion ( (c) )</td>
<td>( kN/m^2 )</td>
<td>4.9 0.4</td>
</tr>
<tr>
<td>3</td>
<td>Unit weight ( (\varrho) )</td>
<td>( kN/m^3 )</td>
<td>15.807 14.288</td>
</tr>
<tr>
<td>4</td>
<td>Liquid limit ( (\text{LL}) )</td>
<td>%</td>
<td>50</td>
</tr>
</tbody>
</table>

Sandy soil composition

Sandy soil ground has to maintain a constant relative density according to the unit weight because the properties of the sand, such as the internal friction angle, vary according to the unit weight. Therefore, sandy soil ground was conducted as follows.

Sand dropping equipment was used to simulate the ground with a uniform density, as shown in Fig. 6(a). The sand dropping equipment consisted of a sand raining sieve and height control equipment. The groove size in the sand raining sieve was 2 \( mm \). The sand dropping equipment was 990 \( mm \) long and 990 \( mm \) wide. The height control equipment can sustain a load of up to 20 \( kN \).

Therefore, sandy soil composition tests were conducted to determine the relationship between the unit weight and drop height using the sand dropping equipment, and to evaluate the unit weight at a height of 10 \( cm \), 20 \( cm \), 30 \( cm \), 40 \( cm \), 50 \( cm \), 60 \( cm \), and 70 \( cm \), as shown in Fig. 6(b). The values of the relative density are calculated by Eq. (1) and Table 1 (Das, 1998). Therefore, based on these test results, the unit weight, relative density and drop height used for the impact test were 1.474 \( kN/m^3 \), 35\% and 300 \( mm \), respectively.
where \( D_r \) is the relative density, \( \gamma_d \) is the dry unit weight, \( \gamma_{d_{\text{max}}} \) is the maximum dry unit weight, and \( \gamma_{d_{\text{min}}} \) is the minimum dry unit weight.

\[
D_r = \left[ \frac{\gamma_d - \gamma_{d_{\text{min}}}}{\gamma_{d_{\text{max}}} - \gamma_{d_{\text{min}}}} \right] \gamma_d
\]  

Fig. 6 (a) Soil dropping equipment and (b) variation of the relative density versus the dropping height.

Clay soil composition

Different clay soils were simulated by changing the moisture content. Clay soils with different shear strengths were used (moisture content = 60\% \( (c = 4.90 \text{ kN/m}^2) \), 80\% \( (c = 0.40 \text{ kN/m}^2) \)). The clay was kneaded with water and allowed to cure in a wet state for 24 hour. The clay foundation was made of approximately 30-mm thick clay layers from the floor to the target height. The foundation was again left to cure for 24 hour.

Test scenario

Sand and clay are distributed widely throughout subsea areas. Therefore, in the experiment, soil comprised of sand and clay, was used to simulate the subsea conditions. The test scenario was designed for the primary aim of determining the behavior of a pipe according to the soil. The foundations were formed from three types to estimate the behavior of the subsea pipelines, as shown in Table 3.

Table 3 Test scenario.

<table>
<thead>
<tr>
<th>No.</th>
<th>Soil condition</th>
<th>Pipe diameter (mm)</th>
<th>Pipe thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Sand + Water</td>
<td>60.0</td>
<td>5.00</td>
</tr>
<tr>
<td>2</td>
<td>Clay (moisture content: 60%)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Clay (moisture content: 80%)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Consideration for in-situ pipe system

All specimens were made from cold drawn seamless mild steel pipes, the grade of which used for the experiment was ASTM A53 Gr.B API 5LB, which is similar to subsea pipe forms. The geometries of the pipe specimen, such as outer diameter, thickness and length were 60.0 mm, 5.00 mm and 1.00 m, respectively. The pipe specimen geometries for the test scenario are also summarized in Table 3. The yield stress and ultimate tensile strength of the material were 243 MPa and 451 MPa, respectively (Mertz et al., 1993).

In the experiment, the ends of the pipe were fixed to the wall of the chamber using a fixing apparatus equipped to the chamber. The fixing apparatus keeps the pipe in place, using bolts and nuts, as shown in Fig. 2. The ends of the pipe were fixed because the pipeline at a certain distance from the region of the collision has the function of protecting settlement, and thus they were utilized as a subsea pipeline. Therefore, a reaction force occurs in the region of impact during a collision with a dropping object.
Experimental results

Direct measurement of dynamic behavior: Strain

A permanent deformation of a pipe depends on the impact velocity and height of the dropping object. In the present study, a 1-m drop height, which is a 5% to 6% permanent deformation of a pipe was determined in accordance with ASME B31.8 (2010). Based on this, an impact velocity of 4.429 m/s was calculated using a finite element analysis, and observed of a 5% permanent deformation of the pipe was observed, as shown in Fig. 7(a). In the impact experiments, however, the actual velocity was measured using a high-speed camera, and showed a 4.34 m/s drop velocity, which is an approximately 5% energy loss during the free fall process. Therefore, to consider the energy loss from the experimental approach, the drop height was set to 1.03 m in all of the impact tests conducted. In the present impact experiment, five tests were conducted for each impact scenario to ensure the repeatability of the test results. In the repeated testing, because the material properties can change through consecutive impacts, the soil was not reused so as to obtain reliable results. Therefore, once each impact test was conducted on the pipe and soil foundation, the pipe and soil were replaced prior to the next impact test.

The results for strains no. 1 and 2 in the sand and clay foundations are represented based on the mean value of the three results, with the exception of the maximum and minimum strain results. The results for strain no. 3 were omitted from this paper because they are similar to the results for strain no. 2, as shown in Fig. 7(b), and the purpose of strain no. 3 was to act as a substitute for strain no. 2 when the deformation of the pipe could not be measured through the vibrations during the collision with a dropping object.

![Graphs showing strain vs. velocity and strain vs. time](image)

Table 4 Maximum strain according to the foundation used in the experiment.

<table>
<thead>
<tr>
<th>Foundation</th>
<th>Strain No. 1</th>
<th>Strain No. 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand + Water</td>
<td>0.01834</td>
<td>-0.01445</td>
</tr>
<tr>
<td>Clay (w=60%)</td>
<td>0.01628</td>
<td>-0.00674</td>
</tr>
<tr>
<td>Clay (w=80%)</td>
<td>0.01966</td>
<td>-0.01098</td>
</tr>
</tbody>
</table>

Table 4 shows the variations in the maximum strain no.1 in sand with water and clay with moisture contents of 60% and 80%. The maximum strain no. 1 remained approximately constant at 0.016 to 0.02, and the variations in strain no. 2 remained constant at -0.015 to -0.006.

In the case of sandy soil, strain no. 1 for a deformation of the pipe was larger than the strain results for 60% moisture content clay and was smaller than the strain result for 80% moisture content clay. Strain no. 2 used for the deformation of the pipe was larger than the strain results from the clay soil. In particular, a large deformation of the pipe was observed in the region of impact because sandy soil provides a strong bearing capacity, and the settlement, where the pipe sank into the foundation, was small.

In the case of the clay soil, strains no. 1 and 2 for the deformation of the pipe in clay with a moisture content of 60% were smaller than the strain results for clay with the 80% moisture content. Strain no. 1 in the 80% moisture content clay showed the largest value. The major explanation for these results is the decreases in cohesion with increase in the moisture content. In particular, the clay could absorb more impact energy in the pipe colliding with a dropping object as the moisture content decreased.
**Time-history of deformation**

A series of impact tests were carried out to examine the behavior of the pipe experimentally. Fig. 8 shows the undeformed configuration and a sequence of three deformed configurations calculated using a high-speed camera and a strain gauge. The variations in the deformation of the pipe are shown with the numbered bullets. Configuration ① shows the condition of the pipe prior to impact. Configurations ②, ③, and ④ show the conditions when increasing the deformation of the pipe over time.

As a result, in the case of sandy soil, a large pipe deformation in the region of impact, and a small settlement for the sandy soil were both observed. Further, a large deformation occurred near the region of impact owing to the large ovalization rate for the pipe. On the other hand, in the case of clay soil, large pipe deformations were primarily observed at the ends of the pipe, and a large settlement for the clay soil was observed. For the clay soil with 60% moisture content, the variations in the deformation of the pipe were the smallest because the clay soil could absorb the impact energy through a strong cohesion. However, the clay soil with 80% moisture content had no bearing capacity or cohesion. Therefore, a large settlement was observed for both the pipe and soil.

Fig. 8 (a) Strains no. 1 and 2 of the pipe and (b) consecutive high-speed photographs of the impact tests in sand and clay.
FINITE ELEMENT ANALYSIS

FE models for the impact tests

In this study, ABAQUS, a three-dimensional finite element analysis program, was used for a comparison with the experimental results and to evaluate the safety of a pipeline after a collision with a dropping object. A finite element model was analyzed as a three-dimensional shape by considering the shape and size in the experiment. Fig. 9(a) shows the FE models used as the impact test specimens, i.e., the pipe, soil, and dropping object. The diameter, thickness, and length of the pipe were determined to be 60.0 mm × 5.00 mm × 1,000 mm, respectively, in the laboratory-scale impact tests.

Fig. 9 (a) FE models and (b) loading and boundary conditions for the impact test specimens.

Fig. 10 Results of the mesh size convergence study according to the (a) longitudinal, (b) thickness, and (c) circumferential directions of the pipe and (d) soil.
The material properties and stress-strain curve for the ASTM A53 Gr. B & API 5LB was applied to the pipe model (Mertz et al., 1993). As one of useful research results regarding the strain-rate dependent mechanical properties of carbon steels, Masaaki and Kozo (2000) investigated strain-rate effect of yield strength and ultimate tensile strength of seven kinds of carbon steels. From the precise investigation, they concluded that strain-rate effect on the yield and tensile strength of carbon steels can be negligible in the range of the strain-rate $1 \times 10^{-3}/s$ and $1 \times 10^{6}/s$. In this regards, the strain-rate dependency of mechanical properties were not considered in the present FE analysis. For the following comparative study, the length, width, and height of the soil specimens were determined to be 1,000 mm $\times$ 1,000 mm $\times$ 800 mm, respectively. The material properties according to the soil types in Tables 1 and 2 were applied to the soil model. The dropping object was also determined using laboratory-scale impact tests.

In addition, the finite element analysis results were significantly affected by the size and number of analytical elements. In the present study, the element convergence studies on the longitudinal, thickness, and circumferential direction of the pipe and soil were conducted, and the optimum number of elements for a finite element analysis was obtained, as shown in Fig. 10. Based on the results, the optimum number of elements in the pipe and soil were determined to be 140,000 and 50,000, respectively. The element type used is an incompatible mode element, i.e., a C3D8I element in ABAQUS.

**Loading, boundary, and contact conditions for the impact tests**

To simulate a collision between a subsea pipeline and a dropping object, the loading, contact, and boundary conditions for the dropping object, pipe, and soil were considered based on the impact tests. In this simulation, interaction between the pipe and soil was applied, and the behavior of the pipe was analyzed. Each condition for the explicit method used in the dynamic analysis was determined as follows.

**Determination of the scale effect of the foundation**

To determine the soil size, the scale effect was considered because a simulated foundation does not have a half-space soil condition (Hight and Leroueil, 2003; Chang et al., 2010). By dropping a heavy weight onto the specimen, the effect of the soil in the impact analysis was examined. Therefore, the values for the displacement and force of the soil were obtained in the simulations using the longest side of the dropped object (L) and various soil sizes (B), as shown in Fig. 11(a). The scale effect test was conducted until constant values of the displacement and force of the soil were reached. As a result, the soil size was determined to be 1,000 mm $\times$ 1,000 mm $\times$ 800 mm (length $\times$ breadth $\times$ height).

![Fig. 11 (a) Simulation and (b) results for the scale effect.](image)

**Loading and boundary conditions**

Fig. 9(b) shows the loading and boundary conditions for the FE mesh. To examine the impact behavior of the pipe and soil, the dropped object fell freely, as in the impact test, the impact velocity and drop height of which were 4.429 m/s and 300 kg.
respectively. The dropping object was assumed to be a rigid body, and its deformation was ignored. In addition, the boundary conditions of the pipe were fixed at both ends, and the four sides of the soil had fixed ends. Owing to its symmetry, half of the geometry was modeled, as shown in Fig. 9(b) (Zeinoddini et al., 2013).

**Contact condition**

A contact condition has various contact models, and of them, the shear friction model was the representative contact model used herein. The shear friction model can be classified into four different types: an isotropic Coulomb friction model, an anisotropic model, an exponential form, and a user-defined model. In this simulation, the most widely used and basic Coulomb friction model was applied. The Coulomb friction, which is provided in ABAQUS for use with all contact analysis capabilities, assumes that no relative motion occurs if the equivalent frictional stress \(\tau_{eq}\) is less than the critical stress \(\tau_{crit}\) (ABAQUS, 2013). In contrast, a slip occurs if the equivalent fictional stress is equal to the critical stress. The equivalent frictional stress and the critical stress can be defined as follows:

\[
\tau_{eq} = \sqrt{\tau_1^2 + \tau_2^2}
\]

\[
\tau_{crit} = \mu \rho
\]

where \(\tau_1\) and \(\tau_2\) are the shear stresses on the contact plane, and \(\mu\) is the friction coefficient. The critical stress is proportional to the contact pressure \(\rho\), as shown in Eq. (3).

In addition, the three friction coefficients were used in this contact model. The friction coefficient for the interaction between the indenter and pipe was 0.6, and the friction coefficients for the interaction between the pipe and soil were 0.3 (steel on sand) and 0.2 (steel on clay) (CRC, 1997; NAVFAC, 1982).

**Material model for the soil**

Numerous failure criteria have been developed. In this simulation, a Mohr-Coulomb failure criterion was adopted to describe the elasto-plastic mechanical behavior of soil in a seabed. In the Mohr-Coulomb criterion, it is postulated that a failure is controlled by the maximum shear stress, which is dependent on the normal stress. This can be represented through a regression Mohr’s circle for the states of stress during a failure in terms of the maximum and minimum principal stresses. The Mohr-Coulomb failure line is the best straight line that meets the Mohr circles, as shown in Fig. 12.

The constitutive model for sand and clay can be described based on the Mohr-Coulomb criterion. This is an elasto-plastic model that uses a yield function of the Mohr-Coulomb form, where the yield function includes the hardening/softening of the isotropic cohesion. On the other hand, the model uses a flow potential with a hyperbolic shape in the meridional stress plane and no corners in the deviatoric stress space. This flow potential is thus completely smooth and provides a unique definition of the plastic flow direction.

![Mohr-Coulomb failure criterion](image-url)
Comparison of the experimental impact tests

Time-history of deformation

Figs. 13 and 14 show the undeformed configuration and a sequence of three calculated deformed configurations. Their positions on the response are identified by numbered bullets, as the experimental result. The variations in the deformation of the pipe at the impact location were observed through a cross section of the pipe, as shown in Figs. 13 and 18.

The simulation results for the time-history deformation are generally similar to the results of the impact test. On the other hand, after the strain values reached the maximum value, they showed different phenomena between the simulation and impact test. The strain values were uniform in the simulation, while the maximum strain values decreased in the impact test. These different results were obtained because of the difference in the elastic spring-back for the pipe and soil. The elastic spring-back of the simulation differs from that of the impact test because of the damping coefficients. Namely, in the results of the impact test, the strain was shown to return to the original conditions. On the other hand, unlike the results for the impact test, this phenomenon could not be observed in the simulation.

![Fig. 13 Sequence of the deformed configurations calculated during impact.](image)

![Fig. 14 Maximum dent depth response of the pipe in the simulation for (a) strain no. 1 and (b) strain no. 2.](image)

Direct measurement of dynamic behavior: Strain and displacement

To verify the effectiveness of a finite element analysis for the simulation results, the simulation results were compared with the results from the impact test. No significant differences were obtained, as shown in Fig. 15 and Table 5.

The simulation results for the dynamic behavior were generally similar to the results of the impact test except for the results for strain no. 1. In the impact test results, the value of strain no. 1 for the 80% moisture content clay was the largest. On the
other hand, in the simulation results, the results of strain no. 1 for sand with water were the largest. These different results were
due to the difference in the format of the model used between the simulation and impact test. While the soil was formed from
tiny grains in the impact test, it was assumed that the soil is a solid in the simulation because it is difficult to simulate soil
composed of tiny grains. Therefore, the soil in the simulation could absorb the impact energy, which was larger than the impact
energy in the impact test.

In addition, Fig. 16 shows the final deformed shape of the pipe in soil with water, and the 60% and 80% moisture content
clay. The displacements of the deformed pipe according to the soil type are also similar except for the displacements in the clay
soil, although a quantitative comparison was not carried out. It is not easy to conduct a quantitative comparison study owing to
the oval geometrical shapes used.

Table 5 Maximum strain according to the foundation used in the FEA and experiment.

<table>
<thead>
<tr>
<th>Foundation</th>
<th>Strain No. 1</th>
<th>Strain No. 2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Experiment (A)</td>
<td>Analysis (B)</td>
</tr>
<tr>
<td>Sand + Water</td>
<td>0.01834</td>
<td>0.01823</td>
</tr>
<tr>
<td>Clay (w=60%)</td>
<td>0.01628</td>
<td>0.01380</td>
</tr>
<tr>
<td>Clay (w=80%)</td>
<td>0.01966</td>
<td>0.01791</td>
</tr>
</tbody>
</table>

Fig. 15 Comparison between the simulation and experiment for strains no. 1 and
2 in (a) sand with water, and (b) 60% and (c) 80% moisture content clay.
Fig. 16 A comparison between the experimental and simulated results of (a) the final shape in (b) sand with water, and (c) 60% and (d) 80% moisture content clay.

Variations in the values for the soil and pipe

Fig. 17 shows the locations for point no. 1, and the variation in the stress of the pipe according to the foundation types. Fig. 18 shows the deformation of the pipe according to the foundation types. The deformation of the pipe in a plate, which is a rigid
As a result, the displacement for point no. 1 increased as the soil softened. Specifically, the displacement for point no. 1 in the 80% moisture content clay was the largest. On the other hand, the deformation of the pipe in sand with water was larger than the deformation in clay. Therefore, it was found that the bearing capacity of the clay was quite low. The stress value for point no. 1 in the 80% moisture content clay was the largest. In contrast, the stress value for the pipe in sandy soil was the lowest. Hence, it was found that the sandy soil could absorb some of the impact energy owing to its elasticity. In addition, the stress in the plate decreased rapidly within 0.015 sec. This phenomenon was attributed to the variation in the pipe form from an oval to a ribbon shape, as shown in Fig. 18(a).

Comparison with ASME B31.8

ASME B31.8 defines plain dents as injurious if they exceed a depth of 5% to 6% of the nominal pipe diameter. These plain dents of any depth are represented by the strain levels associated with the deformation. In addition, they also mentioned that the strain levels do not exceed a strain of 5% to 6%. The strain levels can be estimated using data from the deformation tools or from the direct measurements of the deformation contour in ASME B31.8, Appendix R (ASME B31.8, 2010; Noronha et al., 2010; Rafi et al., 2012; Maziar and Thomas, 2013). On the other hand, the strain levels were not considered for the soil effect. Therefore, this rule was compared with the overall results from the simulation to investigate the precise deformation of the pipe. Fig. 19 shows the method used for estimating the strain in the dents. The strain on the inside (\( \varepsilon_i \)) and outside (\( \varepsilon_o \)) of the pipe surface are defined respectively as follows:
where $\varepsilon_1$ is the bending strain for the circumferential direction, $\varepsilon_2$ is the bending strain for the longitudinal direction and $\varepsilon_3$ is the extensional strain in the longitudinal direction, which can be expressed as follows:

$$
\varepsilon_1 = \left(\frac{1}{2}\right) t \left(\frac{1}{R_0} - \frac{1}{R_1}\right)
$$

$$
\varepsilon_2 = -\left(\frac{1}{2}\right) \frac{t}{R_2}
$$

$$
\varepsilon_3 = \left(\frac{1}{2}\right) \left(\frac{d}{L}\right)^2
$$

where $R_0$ is the initial pipe surface, which is equal to 1/2 the nominal outer diameter, and $R_1$ is the indented outer diameter surface radius of the curvature in a transverse plane through the dent. The dent may only partially flatten the pipe such that the curvature of the pipe surface in the transverse plane is in the same direction as the original surface curvature, in which case, $R_1$ is a positive quantity. On the other hand, $R_1$ is a negative quantity if the dent is a re-entrant, meaning the curvature of the pipe surface in the transverse plane is actually reversed. $R_2$ is the radius of the curvature in a longitudinal plane through the dent. In addition, $t$ is the wall thickness, $d$ is the dent depth, and $L$ is the dent length.
rally larger than the results of ASME B31.8. Therefore, in this study, various factors of the moisture content were investigated, as shown in Eqs. (9) through (12).

\[
\varepsilon_{\text{in}} = \varepsilon_1^2 - \varepsilon_1 (\varepsilon_2 + \varepsilon_3) + (\varepsilon_2 + \varepsilon_3)^2 \left( F_c \right) \tag{9}
\]

\[
\varepsilon_{\text{os}} = \varepsilon_1^2 + \varepsilon_1 (-\varepsilon_2 + \varepsilon_3) + (-\varepsilon_2 + \varepsilon_3)^2 \left( F_c \right) \tag{10}
\]

\[
\varepsilon_{\text{ic}} = \varepsilon_1^2 - \varepsilon_1 (\varepsilon_2 + \varepsilon_3) + (\varepsilon_2 + \varepsilon_3)^2 \left[ F_c + (w\gamma d) / c_s \right] \tag{11}
\]

\[
\varepsilon_{\text{oc}} = \varepsilon_1^2 + \varepsilon_1 (-\varepsilon_2 + \varepsilon_3) + (-\varepsilon_2 + \varepsilon_3)^2 \left[ F_c + (w\gamma d) / c_s \right] \tag{12}
\]

where \( \varepsilon_{\text{in}} \), \( \varepsilon_{\text{os}} \), \( \varepsilon_{\text{ic}} \) and \( \varepsilon_{\text{oc}} \) are the strains on the inner and outer pipe surfaces in the sandy and clay foundations, respectively, \( F_c \) is the failure coefficient according to the location of the strain, \( c_s \) and \( \gamma \) are the cohesion and unit weight for the moisture content, respectively, and \( w \) is the moisture content.

To ensure the safety of a pipeline, the strains for the pipe in ASME B31.8 need to be modified, as shown in Fig. 20(b). Here, \( F_c \) was found to be 1.1 and 1.6 for the inside and outside of the pipe, respectively, based on the differences between the FEA and ASME B31.8.

![Graph showing variations in strain and modified strain according to soil type](image)

**Fig. 20** Variations in (a) strain and (b) modified strain according to the soil type.

**SUMMARY AND CONCLUSIONS**

This paper reported the results of impact testing for the interaction between a pipe and soil, which were simulated as a subsea pipeline and seabed. An impact test was conducted at the foundation according to three types of soil, which were sand with water, and clay with 60% and 80% moisture content. Moreover, simulations including an impact test were carried out. The simulation results were compared with the impact test results to validate the interaction between the pipeline and soil. In addition, to inspect the present code, the simulation results were compared with the results of ASME B31.8. The results have the following general features:
The behaviors of the pipe according to the foundation types showed different results, because sandy soil and clay have different material properties. It was found that sandy soil has a strong bearing capacity and clay has a weak bearing capacity but strong cohesion.

At the foundation of sandy soil, the pipe showed a large deformation at the region of impact and a small settlement for the sandy soil was observed. On the other hand, the pipe showed deformations at its ends, and a large settlement for the clay soil was observed at its foundation.

The FEA results were compared with the results of a laboratory-scale impact test to validate the determined behavior of the pipe according to the foundation types. The simulation results were generally similar to the results of the impact test.

To investigate their applicability to a subsea pipeline analysis, the results of an FEA on the behavior of the pipe were compared with the results of ASME B31.8. The results of the FEA were generally larger than the results of ASME B31.8, however. Therefore, it is essential to consider the interaction between the pipe and soil to examine the safety of a subsea pipeline.

Because this was the first such study conducted on a subsea pipeline, to verify the simulations further, more experiments are needed. The experiments omitted many of the parameters related to a subsea environment and idealized others. In addition, the simulation technology used in the present study will be expanded for application to other extreme environments in a subsea system, as well to other fields in similar environments.

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