In-Situ Deep-Sea Monitoring of Cement Mortar Specimen at a Depth of 3515 m and Changes in Mechanical Properties after Exposure to Deep Sea Condition

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Abstract

The results of the first-ever in-situ monitoring of a large mortar specimen at a depth of 3515 m in the Nankai Trough are presented in this study targeted at creating a technology platform for in-situ monitoring and evaluation of cement-based materials at the seabed to realize deep-sea infrastructures. We successfully monitored in situ the development of strain and hydraulic pressure in the specimen. In addition, the short-term behavior of the specimen can be explained by hydraulic confinement and stress relaxation due to water infiltration. Some contraction strain remained in the specimen even after approximately an exposure to the deep sea condition for one year, causing microstructural damage. The pore entry volume was enhanced toward the center of the specimen, and a decrease in compressive strength and Young’s modulus were observed in the specimen after exposure due to the microstructural damage. Further improvement of the in-situ measurements is required to ensure the waterproofing and pressure resistance of the strain and pressure gauges.

1. Introduction

In recent years, the construction of offshore wind power generation facilities and seafloor diastrophism monitoring platforms besides the pioneering and innovative projects described in our previous report (Takahashi et al. 2021) such as infrastructure for resource development (Miller et al. 2018), carbon dioxide and storage (Hume 2018), and next-generation neutrino telescopes (Le Breton 2019), have been continuously conducted for the utilization of deep-sea areas. In 2020 in Japan, the Ministry of Land, Infrastructure, Transport, and Tourism has established the technical standards and guidelines for the construction of offshore wind power generation facilities, and its commercialization is rapidly progressing. Floating facilities provide more access to deeper sea areas at depths of 100–300 m in which cement-based materials can be used for their platforms and anchors. Carbon dioxide (CO₂) emissions can be reduced by half using a combination of steel and concrete composite structure rather than using steel alone (Choisnet et al. 2016). In addition, support structure designs with cement concretes have received attention owing to their turbine size and application to deeper sea areas (Mathern et al. 2021). Moreover, a trial has been started to monitor the real-time crustal deformation at the ocean bottom as part of the project for national resilience by the Ministry of Education, Culture, Sports, Science and Technology, enabling the discussion of rupture regions of future mega-thrust earthquakes (Kimura et al. 2021). For this trial, an anti-washout cement mortar with a long handling time was used to fix the tiltmeters and hydrometers at the seabed to improve measurement accuracy.

It is essential to obtain data and insights on how the materials behave after long-term exposure to deep sea conditions when considering the aforementioned applications of cementitious materials in the deep sea. The authors reported the chemical disintegration of cement mortars exposed to the deep-sea floor and interpreted that the key influencing factor in the deterioration is extremely low temperature (Kobayashi et al. 2021). Another key factor is a high hydraulic pressure. In-situ monitoring of the effect of high hydraulic pressure re-
Table 1 Mix proportions of the experimental mortar (unit: g). Siliceous sand with a particle size of 0.1–2.0 mm, which is standard sand produced by Japan Cement Association, was used.

|      | Cement | Sand | PCE  | MC  | Defoamer | Tap water |
|------|--------|------|------|-----|----------|-----------|
| 500  | 500    | 0.5  | 1.0  | 0.1 | 300      |           |

Table 2 Composition of high-early-strength Portland cement measured by X-ray fluorescence (unit: %).

|         | SiO₂ | Al₂O₃ | Fe₂O₃ | CaO | MgO | SO₃ | Na₂O | K₂O | LOI | Total |
|---------|------|-------|-------|-----|-----|-----|------|-----|-----|-------|
| 20.82   | 5.10 | 2.34  | 66.13 | 1.13| 2.60| 0.21| 0.26 | 0.92| 99.51|

*LOI: loss on ignition

Table 3 Physical properties of the experimental mortar. Flow value, air content, compressive strength, Young’s modulus, and bending strength were measured based on JIS A 1108: 2018 and JIS A 1149: 2017.

| Flow value (mm) | Air content (%) | Density (g/cm³) | Compressive strength (N/mm²) | Young’s modulus (kN/mm²) | Bending strength (N/mm²) |
|----------------|-----------------|-----------------|-----------------------------|--------------------------|--------------------------|
| 262            | 3.5             | 1.93            | 34.2                        | 15.56                    | 5.3                      |

veals real structural damage to cementitious materials under deep-sea conditions, although these studies are rarely reported owing to the lack of large-scale equipment reproducing actual deep-sea ocean environments (temperature, pressure, currents, seawater, etc.), difficulty in accessing the actual deep sea, and no in-situ testing equipment with waterproof and high-pressure resistant specifications. Kawabata et al. (2022) recently developed a new apparatus to observe the internal damage, microstructure, and deformation of the specimen under hydraulic pressure using an X-ray microcomputed tomography scanner and measured water infiltration into a small mortar specimen (diameter of 50 mm and length of 100 mm) and volumetric changes of the specimen under short-term high hydraulic pressure. The action of hydraulic pressure has a size effect, and a larger specimen may exhibit more severe damage. Therefore, the significance of this study is to provide the testing results of the first-ever in-situ monitoring at the deep-sea floor using a large specimen with diameter 350 mm and a length of 400 mm.

With a view toward deep-sea infrastructure construction, the authors have set up a project targeted at creating a technology platform with in-situ methods and systems for monitoring and evaluating cement-based materials located at deep ocean bottom sites. In this study, in-situ changes in strain and hydraulic pressure in a large cylindrical mortar specimen exposed at a depth of 3515 m in the Pacific Ocean’s Nankai Trough for 309 days. In addition, we measured the mechanical properties of the specimen after an exposure of 309 days, including the compressive strength, Young’s modulus, pore entry volume, and microhardness; their correlations with the in-situ monitoring data were discussed.

2. Experimental

2.1 Specimen preparation

Table 1 shows that the experimental mortar mix contains high early strength Portland cement, siliceous sand, polycarboxylic ether superplasticizer (PCE), methyl cellulose (MC) thickener, and defoamer. Table 2 lists the composition of the cement, which was measured by X-ray fluorescence according to the Japanese Industrial Standard (JIS) R 5304. The dry mortar mix was pre-mixed and then mixed again with water for 3 min at 1100 rpm using a hand-held mechanical mixer. Subsequently, the mortar was poured into a cylindrical mold with a diameter base of 350 mm and a height of 400 mm. Prior to the in-situ deep-sea exposure, the mortar specimens were cured for 28 days under sealed conditions at 20°C. There was a ± 1°C variation in the curing temperature.

This mold size allowed for three strain gauges PML-60 and three pressure gauges PWF-50MPB (Tokyo Measuring Instruments Laboratory Co., Ltd. (TML)) to be embedded, as shown in Fig. 1. The head of the pressure gauge has a diameter of 14.8 mm and a length of 51 mm. The side and loading surfaces were covered with a rubber sheet of thickness 1 mm and permeable melamine foam, respectively, to detect only the hydraulic pressure while mitigating the constraint from the side and loading surfaces of the pressure gauge (see Fig. A1). All lead wires were covered with waterproofing heat-shrink tubing SUMITUBE W3C (Sumitomo Electric Industries, Ltd.). The strain gauges were flanked in the specimen by one at 175, 88, and 44 mm in the radial direction from the curved side surface of the specimen (locations described as the center, 1/2r, and 1/4r, respectively). In addition, pressure gauges were embedded 80 mm depth from each base of the specimen, which were situated contiguous to the center, at 1/2r, and at 1/4r. The surfaces of the upper and lower bases of the cylindrical mortar specimens were coated with water-impervious epoxy resin to monitor the effects of water infiltration from the side surface.

Table 3 lists the flow value, air content and density of the mortar immediately after mixing, compressive strength, Young’s modulus, and bending strength of the mortar specimen cured for 28 days under a sealed condition at 20°C.
2.2 In-situ deep-sea test

The apparatus designed for in-situ measurements are described in our previous study (Takahashi et al. 2021). The mortar specimen and apparatus were set on the seafloor at the deep-sea research site with a depth of 3515 m by the remotely operated vehicle (ROV) KAIKO MK-IV shown in Fig. 2 during the KR 20–07 cruise. The descending speed of the ROV was approximately 50 m/min, which was equivalent to the pressurization rate of 0.5 MPa/min. Moreover, the temperature, salinity, and dissolved oxygen of seawater at the experimental site when the specimen was set at and retrieved were 1.5°C, 35 PSU, and 3 mL/L, respectively.

During the KR 21–07 cruise, the mortar specimen was retrieved from the experimental site using the same ROV. The suspension rope attached to the stainless-steel frame of the mortar specimen was grabbed with a manipulator of the ROV, as shown in Fig. 3, and the specimen and measurement unit were carefully picked up and stored in the basket of the ROV, as shown in Figs. 4 and 5. These operations were monitored by video cameras installed onboard the ROV. After leaving the seafloor, the ROV rose to the sea surface after approximately 2 h (i.e., the depressurization rate was approximately 30 m/min or 0.3 MPa/min). The hydraulic pressure and strain exerted on the mortar specimen were measured continuously from 25 August 2020 to 30 June 2021 (309 days). Data were acquired once every 3 min for the first two days and then recorded once every 24 h afterward. Each measurement was conducted only once with one mortar specimen due to several limitations, such as difficulties in using research ships.

2.3 Mechanical properties of the mortar core after exposure

Figure 6 shows the mortar specimen retrieved from the deep-sea bottom. The cores were drilled from the specimen in the lateral and longitudinal directions to avoid embedded gauges. The cores drilled from the longitudinal direction shown in Fig. 7 were used to test the compressive strength and Young's modulus. Conversely, the cores drilled from the lateral direction were used to measure the pore entry volume and microhardness using mercury intrusion porosimetry (MIP) and Vickers test.

Fig. 1 Schematic of the embedded strain (red) and pressure gauges (blue) in the cylindrical mortar specimen (unit: mm).

Fig. 2 ROV KAIKO MK-IV used for the in-situ deep sea test.

Fig. 3 Operation using a manipulator of the ROV to grab the suspension rope attached to the stainless-steel frame of the mortar specimen.
respectively. Figure 8 shows the preparation of these test specimens.

The compressive strength and Young’s modulus were measured according to JIS A 1108: 2018 and JIS A 1149: 2017. The drilled cores, which has a diameter of 50 mm and a length of 120 mm, were cut off 10 mm at the top and bottom ends. Three core specimens were prepared.

MIP was performed using the AutoPore IV 9500 system (Micromeritics Instrument Corporation) with a contact angle of 140° at a temperature of 25°C. Although MIP cannot measure the actual pore sizes, it can be used to determine the comparative indices for the pore capacity where mercury can penetrate beyond the
threshold pressure (Diamond 2000). The drilled cores were sliced at 0–5, 5–10, 10–20, 20–30, 30–40, 70–80, 110–120, and 165–175 mm from the surface to the center of the specimen. The sliced specimens were crushed until they could pass through a 5 mm sieve and were retained on a 2 mm sieve. Subsequently, they were dried through solvent exchange with isopropanol for seven days to remove water, and then put in a fume hood for 3 h and in a desiccator with silica gel for seven days to evaporate the water–alcohol mixture (Scrivener et al. 2015).

The Vickers test was performed using the drilled core based on JIS Z 2244-1: 2020. The measured distances from the specimen surface were 1, 3, 5, 10, 20, and 100 mm, and ten points were tested at each distance.

3. Results and discussions

3.1 Visible damage appearance

There is no observed severe damage, except for the corner of the specimen, as shown in Figs. 6 and 9. In contrast to the previous study (Kobayashi et al. 2021), the specimen in this study retained its original cylindrical shape. This difference could be attributed to the size effect on ion dissolution. Hydrates, such as calcium hydroxide, could be stable even at the specimen surface where seawater contacts as a sufficiently high pH was maintained in the large-size specimen.

Figure 9 shows the close-up view of the damaged corner of the retrieved mortar specimen. At the corner area, the specimen at approximately 3 cm in depth was very fragile, appeared crushed and bruised, and easily disintegrated at touch. This damage was not caused by the pressure release during the ascent of the specimen. In a previous study, the authors reported that cracks were initiated at the corners of the specimen when hydraulic pressure was applied for a few hours (Takahashi et al. 2021). Here, the disintegration of the microstructure could be enhanced due to ion dissolution and the formation of ettringite, thaumasite, and Mg-based hydrates (Kobayashi et al. 2021). Cracks may occur while the specimen descended and/or shortly after the specimen reached the deep-sea floor.

3.2 Strain and pressure measured in-situ at deep sea bottom

3.2.1 Precision of the monitoring data and temperature compensation

The data variation of the strain and pressure was checked using the retrieved specimen to ensure the precision of the in-situ monitoring data shown in Figs. 10 and 11. The variation of the strain was confirmed by monitoring the changes in the strain values for 60 s. All data were logged within ± 1 μ, and this result revealed...
that there had been no data variation during in-situ monitoring of the strain at the deep-sea bottom. The strain and pressure values could be increased or decreased even when the gauges were load-free when data variation occurred during deep-sea exposure. One reason for the data variation is the moisture uptake on the gauge base. Conversely, high variation in pressure data was observed after three days of exposure. In addition, the breakdown of pressure gauges was confirmed by the insulation check using the specimen after exposure. Moreover, the pressure gauge embedded in the center of the specimen exhibited a large negative pressure immediately after the beginning of the measurement. Therefore, the changes in pressure at 1/2r and 1/4r during the initial 24 h are plotted in Fig. 10.

A thermal shrinkage of 111 µ appears due to the temperature decrease on the ship deck and the seafloor (30°C to 2°C) based on the temperature compensation data sheet provided by TML, which is specific to the strain gauge used (see Appendix A1). As this thermal shrinkage of the strain gauge is maximum 2–3% of the actual shrinkage generated in the mortar specimen at the seabed (several thousand micrometers). Thus, the strain values in this study are expressed as the shrinkage including the temperature influence of the strain gauge. The pressure values were temperature compensated in the circuitry of the pressure gauge.

3.2.2 Strain and pressure monitored in situ during the initial 24 h

Figure 10 shows the changes in strain and pressure in the mortar specimen during its descent and after the exposure to the deep-sea floor at the depth of 3515 m for the initial 24 h. The plot of the ambient hydraulic pressure presented in Fig. 10 was calculated from the diving depth of the ROV measured by a depth meter. All strains at the center, 1/2r, and 1/4r decreased (the specimen contracted) with increasing applied hydraulic pressure and showed the same value during the descending. The applied pressure was assumed to be constant at approximately 35 MPa when the specimen reached a depth of 3515 m. Immediately after the ambient hydraulic pressure became constant, the pressure in the specimen was detected at 1/4r, and the strain increased (the contracted specimen was restored) with an increase in the pressure value in the specimen. Based on the principle of effective stress by Terzaghi (1943), the effective stress is generated in the skeleton of the specimen when there is no liquid water in the specimen and an external hydraulic pressure is applied. Subsequently, the specimen is saturated with water, and the water in the pores resists the hydraulic pressure without effective stress development of the skeleton. This principle is generally applied to saturated soil mechanics, but should be considered for unsaturated specimens (Fredlund, 2006). Some pores in the specimen should be unsaturated as the mortar specimen used was hydrated for 28 days before exposure. Therefore, the skeleton should resist the applied hydraulic pressure. Subsequently, when water penetrated the specimen, the water in the pores resisted the applied hydraulic pressure and the effective stress in the skeleton was relaxed, increasing the strain.

At 1/2r, the strain continued to decrease for a short period after the specimen reached the seabed. Creep might occur in this short period due to the confinement by hydraulic pressure, resulting in the continuous decrease of the strain. The creep behavior could be counteracted when stress relaxation by water infiltration started. After 6–7 h, the hydraulic pressure was detected at 1/2r, and the contraction strain was relaxed with increasing pressure due to water infiltration. At the center,
although the defect pressure gauge did not show the onset of the pressure, the relaxation of the strain curve at the center suggested that the pressure increase at the center of the specimen probably started at approximately 8–12 h. Water could reach the center faster than 1/2r since the contraction strain at 1/2r was larger than the strain at the center. The may potentially be due to the heterogeneity of water infiltration by the gradient of the hydraulic pressure derived from the irregularity of pore formation in cementitious materials.

The pressure at 1/2r increased rapidly and then decreased at approximately 12 h, indicating that water might penetrate rapidly due to crack occurrence, and the area around the crack was gradually saturated with water. At 18 h, the pressure at both 1/2r and 1/4r exceeded 35 MPa (hydraulic pressure at a depth of 3515 m). It is possible that the mortar constrained by the hydraulic pressure pushed the side surface of the pressure gauge, bending the loading surface of the gauge (see Fig. A1).

Then, the strain gauge at the loading surface is stretched, which converts to a positive pressure value. Therefore, a higher value was plotted than the actual pressure value. Note that less confining pressure was detected by the pressure gauge embedded in the mortar in seawater under ambient pressure. The details are provided in Appendix A1.

Then, the differences between the results presented in Fig. 10 and the results of the preliminary lab test shown in Takahashi et al. (2021) using the same size mortar specimen with a hyperbaric chamber are discussed. In Fig. 10, the strains were approximately –2700 μ and were smaller (showed larger contraction) than the preliminary result, which showed approximately –1800 μ (see Fig. A4), when the applied pressure reached 35 MPa. Moreover, although all strain curves converged to a constant value in 3 h in the previous report, the strain curves presented in Fig. 10 did not converge to a constant value even after 24 h. This result could be derived from the difference in the denseness of the mortar specimens and/or due to the small difference in pressure increase rates (0.5 MPa/min in this study vs. 0.6 MPa/min in the preliminary study). Figure A5 shows that the pore entry size of each intrusion step was higher in the specimen used in this study than in the preliminary study, which might be derived from the raw material lots. The air content was lower, and the density was higher in the specimen used in this study than in the previous report, as shown in Table A1. In addition, water infiltration becomes slower and strain relaxation takes longer when the specimens are densified. Furthermore, the localization of strain development is more likely to occur owing to heterogeneous water infiltration in cementitious materials derived from discontinuity of pores and heterogeneity of pore distribution (Kawabata et al. 2022). The contraction strain should be kept higher when the stress is not relieved. Tanaka et al. (2020) and Hori et al. (2015) reported that the effects of hydraulic pressure on the strain development and the corresponding damages at microscales and macroscales depend on the pore volume and strength of the specimen. The interpretation of Fig. A4 described in Appendix A2 supports that liquid water barely infiltrated the specimen in this study, but some infiltrated in the specimen of the previous study.

3.2.3 Changes in strain monitored in situ during an exposure of approximately 1 year

Strain curves converged to constant values after a day, as shown in Fig. 11. Theoretically, the curves should converge at the same value as the result of the preliminary lab testing. However, the value presented in Fig. 11 was the smallest at 1/4r, followed by 1/2r and finally the center. At the outer part (1/4r), stress could be relieved due to water infiltration and the strain converged to approximately ~800 μ, which was similar to the preliminary result. Conversely, at the inner part (1/2r and center), stress was not relaxed compared to the outer part even after 1 year has elapsed. In addition, the strain at the center gradually decreased. The difference in these strain curves is consistent with the depth profile of microstructural damages measured by MIP, as described in Section 3.4. Moreover, a gradual decrease in strain was observed at 1/4r from 150 to 180 days. Although pores are saturated because of water infiltration, as previously mentioned, nanoscale pores cannot be completely saturated with liquid water. The hydraulic confinement of the mortar specimen at the seabed may cause damages due to creep around the nanoscale unsaturated pores (Pichler and Lackner 2008; Vandamme and Ulm 2013).

3.3 Compressive strength and Young’s modulus of the specimen after exposure

Table 4 lists the compressive strength, Young’s modulus, and density of the core specimens drilled from the longitudinal direction of the mortar after the deep-sea exposure. The mortar specimen cured for the same period (309 days) with a sealed cover at a temperature of 20°C and atmospheric pressure of 0.1 MPa was referred to as a control. Figure 12 shows stress-strain curves generated from three control specimens and three core specimens, and the deviation between the same samples was very small. Both the compressive strength and Young’s modulus were smaller in the core specimen than the control specimen. The decrease in Young’s modulus was more apparent compared to the decrease in the compressive strength, indicating the occurrence of microstructural damages in the specimen after exposure at the seabed. Furthermore, chemical deterioration would not affect the decrease in compressive strength and Young’s modulus since the top and bottom 10 mm of the specimen, which were severely degraded due to chemical deterioration, were cut-off for these measurements (see Fig. 8).

Table 4 shows that 59.2% of Young’s modulus decrease and 27.7% of compressive strength decrease occurred in the mortar specimen used in this study, and the
compressive strength was smaller than the result from Kobayashi et al. (2021), where approximately 75% of strength decrease in the cuboid specimen with a dimension of 40 × 40 × 160 mm was shown. It takes a longer time for the specimen to be sufficiently saturated when the size of the specimen is large, and the corresponding damage by hydraulic confinement resulting in the loss of strength becomes more apparent. Conversely, degradation due to seawater attack and ion dissolution becomes less apparent due to sufficient alkali sources in the specimen. As the latter effect could be more dominant than the former, the strength loss of the specimen in this study became smaller.

3.4 MIP and Vickers hardness of the specimen after exposure

Figures 13 and 14 show the incremental pore volumes of the mortar specimens after exposure to the seabed.
There were two main intrusion steps except for the 0–5 mm area: the capillary pore entry corresponding to the largest peak of the derivatives at approximately 20–100 nm and a small pore entry at less than 20 nm. Because the small pore entry does not significantly differ in the depth direction and is not favorable for discussion solely by the MIP (Olson et al. 1997), the depth profile of the capillary pore entry was compared. The threshold diameter of the capillary pore entry increased from the 5 to 10 mm area toward the center (165–175 mm) as shown in Fig. 13, indicating that the microstructure of the specimen gradually became coarse toward the center. Hydrates and/or seawater-derived products (e.g., magnesium hydroxide) may block the water path and contribute to densification near the surface. It is possible that the larger strain remained at the skeleton and the damage to the microstructure became more severe toward the center, as the stress relaxation should be prolonged and/or less pronounced at the areas closer to the center than at the surface. A larger strain at the center compared to that at the near surface (1/4r) was observed in Fig. 11. Therefore, it can be concluded that the MIP result is consistent with the results of the in-situ strain measurement.

Conversely, Fig. 14 shows that the capillary pore entry was notably enlarged in the vicinity of the surface, while the threshold diameter in the 0–5 mm area was approximately 1000 nm in contrast to the diameter of approximately 100 nm in the 5–10 mm area. Near the specimen surface that is in contact with seawater, the combination of significant alkali leaching, ettringite formation, and formation of reactants with low binding capacity could coarsen the pore structure. Figure 15 shows the Vickers hardness as a function of the distance from the specimen surface. The Vickers hardness from the surface to a depth of 5 mm was lower.
than that at other depths, which was similar to the result of MIP. However, there was less difference between 10, 20, and 100 mm and the control specimen. Vickers hardness is affected by macroscopic damage although MIP can measure these damages. Therefore, less difference appeared in the hardness deeper than 10 mm.

4. Conclusion

The main conclusions found from our in-situ deep-sea tests conducted at a depth of 3515 m are the following:

(1) The development of the strain and pressure was successfully monitored in situ. The initial behavior can be explained by the confinement of the hydraulic pressure and stress relaxation due to water infiltration.

(2) The confinement near the specimen surface was relieved soon, but contraction strain remained in the areas closer to the center of the specimen even after approximately one year of deep-sea exposure. Strain development during an exposure of one year was consistent with the depth profile of the microstructural damage by MIP, indicating that the microstructure became coarse toward the center of the specimen.

(3) Hydraulic confinement during an exposure of one year may cause damages due to creep.

(4) A decrease in the compressive strength and Young’s modulus was observed in the specimen after exposure.

(5) The size of the mortar specimen could affect its physicochemical properties. In this study, the larger specimen used retained its original shape.

(6) The accuracy of in-situ measurements can be improved by ensuring the waterproofing and pressure resistance of the gauges.

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Appendix
A1. Outlook for the improvement of in-situ measurements

The pressure and strain gauges should be improved for further in-situ measurements based on the results of the first in-situ trial at the seabed. The pressure gauge used was not designed for utilization under deep sea where very high pressure acts on gauge parts. As described in Section 2.1, the side surface of the pressure gauge was covered with a rubber sheet with a thickness of 1 mm to avoid contracting pressure from the compressed mortar by hydraulic pressure. However, this may not be sufficient, as interpreted in Section 3.2.2. Figure A1 illustrates the schematic image of the pressure gauge and possible deformation due to hydraulic pressure. Figure A2 shows a comparison of the thickness of the rubber sheets covering the side surface of the pressure gauge. Prior to testing, the pressure gauge was embedded in a mortar with a diameter of 80 mm and a height of 100 mm, which was sufficiently larger than the pressure gauge, and cured for 28 days. A high-pressure vessel with an inner diameter of 150 mm and a height of 250 mm was used for pressurization (Takahashi et al. 2021). In the case of the 1 mm thickness used in this study, the reading value was 1.16 times higher than the applied pressure. Conversely, in the case of 3 mm thickness, the reading value was closer to the applied pressure, although it was still 1.06 times higher. The reason why the reading value becomes higher than the actual hydraulic pressure is because the loading surface is further concave inward, the stretched strain gauge at the loading surface converts to a positive pressure value where the total reading pressure increases when the loading surface is pressed from the side surface while the loading surface is pressed by the hydraulic pressure. This improvement can be applied to future in-situ measurements.

Figure A3 shows the seawater depth, temperature, and corresponding thermal strain of the strain gauge as a function of time. The thermal strain was calculated using the temperature compensation data sheet provided by the TML. Although the thermal strain was very small compared to the strain caused by hydraulic pressure described in Appendix Fig. A3, the temperature compensation of the strain gauge should be considered by

\[ y = 1.1629x \]
\[ R^2 = 0.9912 \]

\[ y = 1.0606x \]
\[ R^2 = 0.9947 \]
measuring the temperature gradient in the specimen for an elaborate discussion of the effect of the hydraulic pressure.

The lead wire was covered with heat-shrink tubing for waterproofing in this study, and the covered wire enabled in-situ strain measurement for approximately a year. However, further long-term in-situ measurements would require improvement of waterproofing. It may be necessary to wrap the lead wire with butyl tape, then with self-bonding tape, followed by covering them with waterproofing heat-shrink tubing.

A2. Strain as a function of applied hydraulic pressure

Figure A4 plots the strain measured with an embedded strain gauge in a sealed mortar specimen as a function of applied hydraulic pressure. The mortar specimen, which has a size of $40 \times 40 \times 160$ mm, was sealed with a water-impervious epoxy resin. A high-pressure vessel with an inner diameter of 150 mm and a height of 250 mm was used for pressurization presented in Takahashi et al. (2021). The strain data during the descent to the seabed that was obtained from the in-situ measurement shown in Fig. 10 and from the preliminary laboratory testing shown in Takahashi et al. (2021) were also plotted in Fig. A4. The strain of the sealed mortar was decreased linearly as a function of the applied pressure and showed almost similar trend with that of the in-situ data of the mortar specimen descending to the seabed. The data plots from the laboratory testing also showed a similar trend with them until the applied pressure reached 15 MPa.

\begin{equation}
\text{Sealed mortar} \quad y = -80.83x \\
R^2 = 0.9993
\end{equation}

Fig. A3 Seawater depth, temperature, and the corresponding thermal strain of the strain gauge.

Fig. A4 Strain measured by the strain gauge embedded in a sealed mortar specimen as a function of applied hydraulic pressure. Strain data during the descent to the seabed obtained from the in-situ measurement shown in Fig. 10, referred to as in-situ, and from the preliminary laboratory testing shown in Takahashi et al. (2021), referred to as lab, were also plotted for comparison.
A3. Comparison with the previously reported data

Figure A5 represents the difference in the pore volume by MIP between the specimen of Takahashi et al. (2021) and the specimen in this study. The specimens were cured for 28 days with a sealed cover at 20°C. Table A1 lists the differences in the flow value, air content, density, compressive strength, and bending strength of the mortars used in the study of Takahashi et al. (2021) and this study.

|                     | Flow value (mm) | Air content (%) | Density (g/cm³) | Compressive strength (N/mm²) | Bending strength (N/mm²) |
|---------------------|-----------------|-----------------|-----------------|-----------------------------|--------------------------|
| Takahashi et al. 2021 | 268             | 5.0             | 1.90            | 34.0                        | 5.8                       |
| This study          | 262             | 3.5             | 1.93            | 34.2                        | 5.3                       |

Fig. A5 Difference in pore volume by MIP between the specimen of Takahashi et al. (2021) and this study.