Survey of the structure of Pit No. 3 winding machine room (Survey of Hashima Coal Mine buildings, etc.)

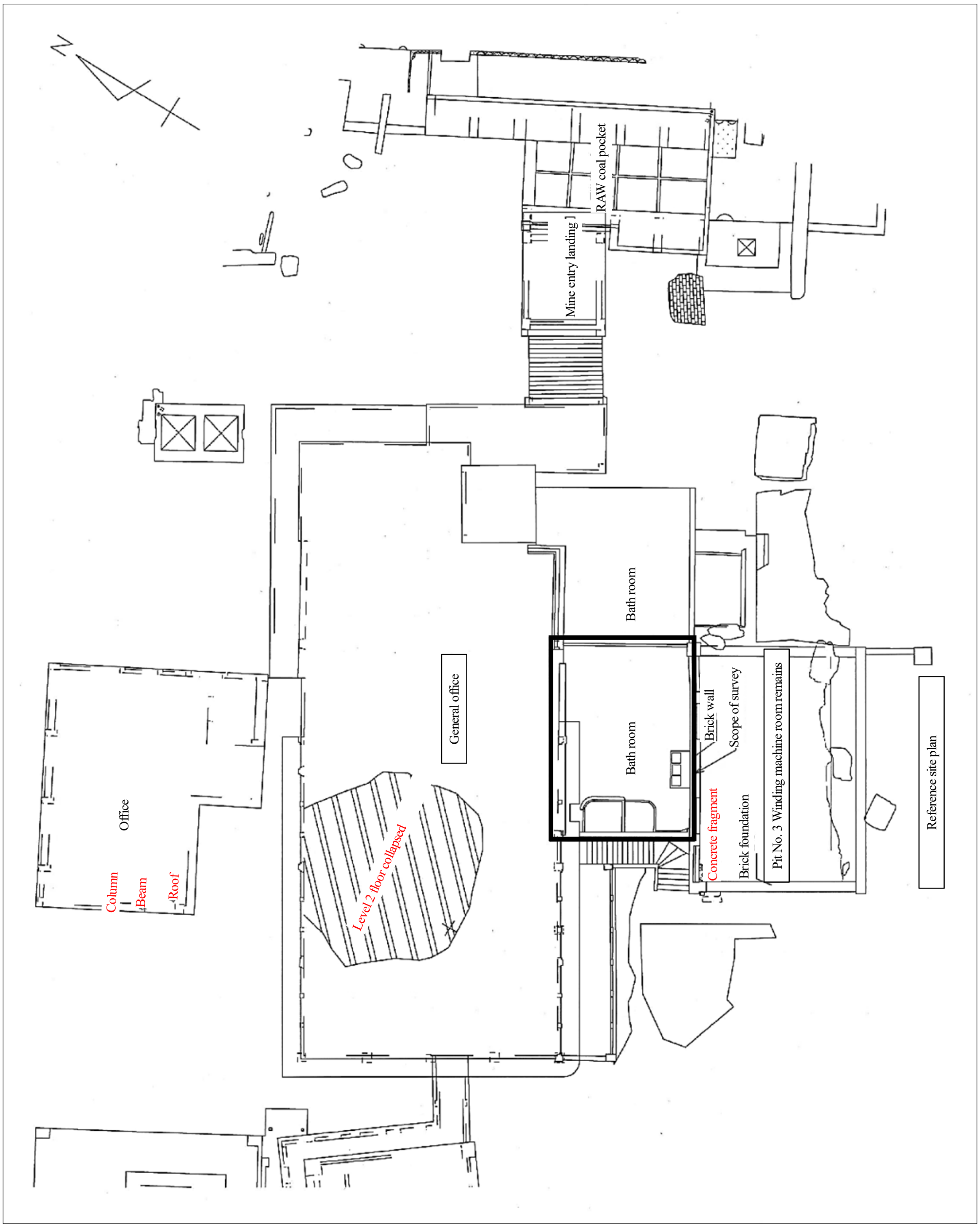


Figure 2-4-33 Survey locations inside building

Appendix 3

1) Content of survey

① Measurements

The following measurement surveys were conducted:

1. Measurements required to create site plans and measure the land subsidence
2. 3D laser measurements required to create drawings of the current conditions of the building

   Laser measurements were taken using a scanning pitch of 7 mm or smaller per 10 m.
3. Rubbles (excluding those that could not be moved manually), wooden pieces, steel shelves, and temporary scaffolding materials that would interfere with 3D laser measurement were moved to the adjacent office. Moving of such objects, however, was kept to minimum in order to preserve the current conditions as much as possible.

② Investigation of damage

The outer walls, floors, inner walls, columns, beams, ceilings, and roofs were examined for damage (cracks, floating, concrete cover falling off, rebars exposed, rebars lost, mortar finishes floating, etc.), visually and by tapping on the surface. Investigation was also made to examine the joint between the brick walls and the General Office, cracked walls inside Arch No. 4 of brick walls, and rainwater infiltration at the top of the brick walls.

③ Creation of drawings of current conditions

Based on the results of measurement surveys, a series of line drawings of the current conditions of the site were created. A list of drawings created is provided in Table 2-4-30. The southern side elevation plan was compared to the drawings that had been created based on measurements taken in FY2015 survey.

④ Creation of drawings of damage

1. Based on the results of the investigation of damage, the conditions of damage were summarized in the drawings of the current conditions to create drawings of damage. A list of drawings created is provided in Table 2-4-30.
2. The conditions of damage were documented in photographs in the following manner:
   a) Pictures of an object were taken alongside a whiteboard that contained the name of the task, date, and location on it. As an exception, close-up pictures were taken without a whiteboard in them, by preparing a diagram indicating the locations of the images.
   b) Pictures were taken in parallel or perpendicular to the object.
   c) Photo resolution: Approximately 2 megapixels (1204 × 1606); 300 dpi

Table 2-4-30. List of diagrams and plans (Drawings of current conditions / damage)

No	Name of diagram/plan	Scale	Category	Drawings of current conditions	Drawings of damage	Remarks
01	Site plan	1/300	Site plan	○		A-01
02	Diagram of land subsidence measurements (Shown in site plan)	1/300	Site plan	○		
03	Level 1 plan view	1/50	Plan view	○	○	A-02
04	Diagram of land subsidence measurements (Shown in plan view)	1/50	Plan view	○		
05	Level 2 plan view	1/50	Plan view	○	○	A-03
06	Northern side elevation view	1/50	Elevation view	○	○	A-04
07	Western side elevation view	1/50	Elevation view	○	○	A-05
08	Southern side elevation view	1/50	Elevation view	○		A-06
09	Cross-section view (1)	1/50	Cross-section view	○		A-07
10	Cross-section view (2)	1/50	Cross-section view	○		A-08
11	Level 1 developed view (1)	1/50	Developed view	○	○	A-09
12	Level 1 developed view (2)	1/50	Developed view	○	○	A-10
13	Level 1 developed view (3)	1/50	Developed view	○	○	A-11
14	Level 2 developed view (1)	1/50	Developed view	○	○	A-12
15	Level 2 developed view (2)	1/50	Developed view	○	○	A-13
16	Level 1 reflected ceiling plan	1/50	Plan view	○	○	A-14
17	Beam developed key plan	1/50	Plan view	○		A-14a
18	Level 1 ceiling beam developed view (1)	1/50	Developed view	○	○	A-15
19	Level 1 ceiling beam developed view (2)	1/50	Developed view	○	○	A-16
20	Level 1 ceiling beam developed view (3)	1/50	Developed view	○	○	A-17
21	Level 2 eaves framing plan	1/50	Framing plan	○	○	A-18
22	Comparison of southern side developed plans	1/50	Elevation view	○		A-19

Appendix 3

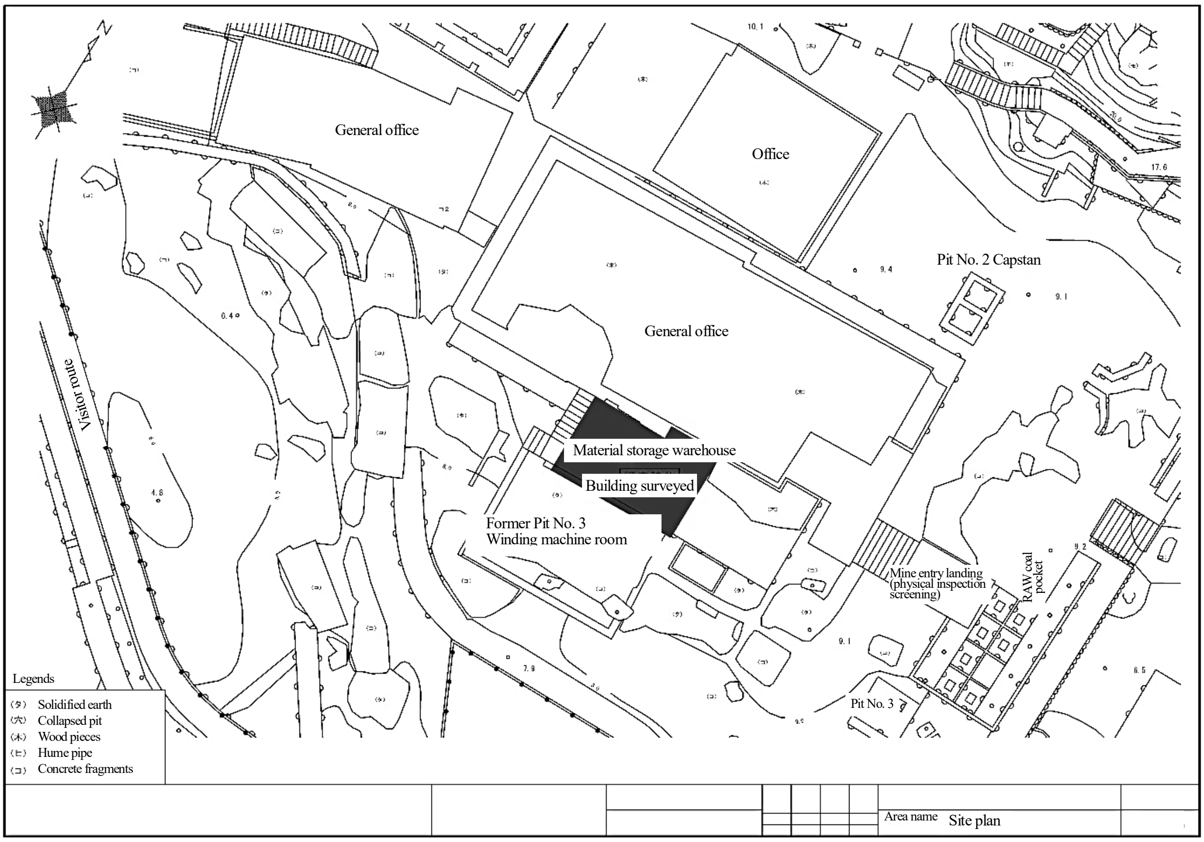


Figure 2-4-34. Site plan

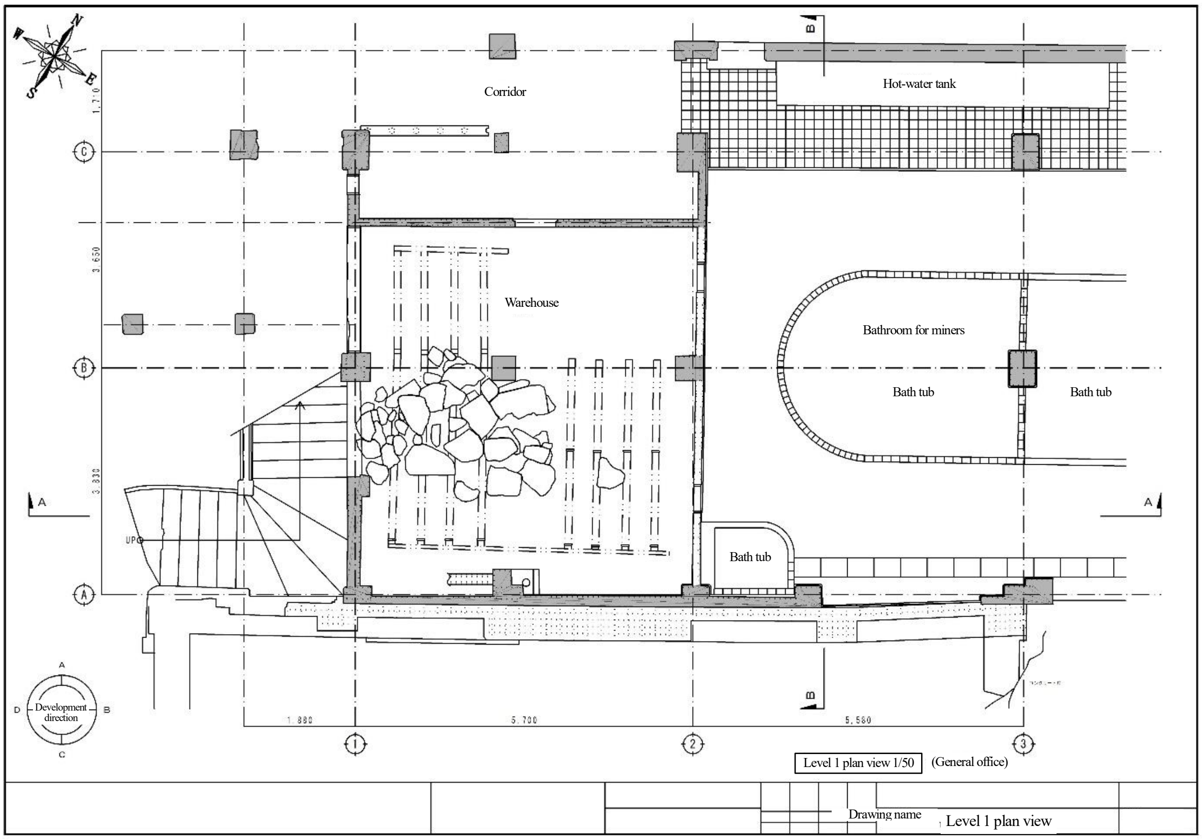


Figure 2-4-35. Level 1 plan view

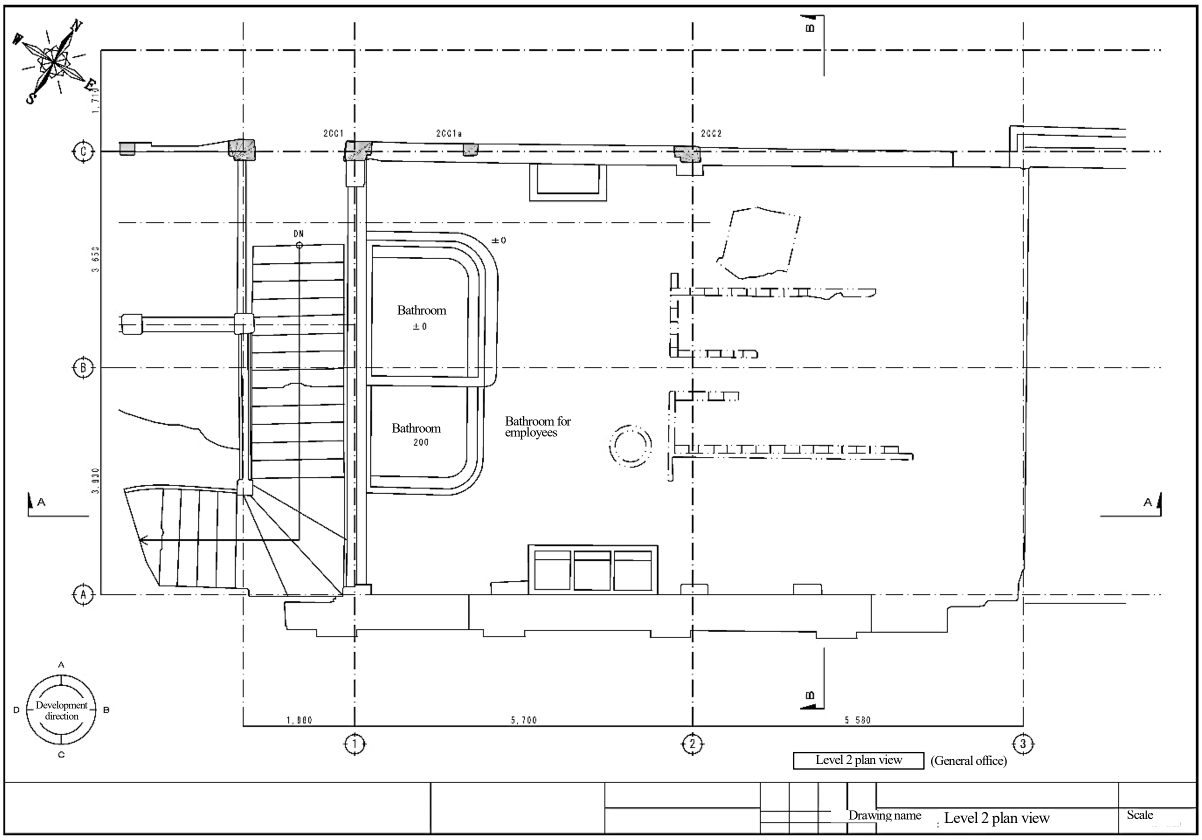


Figure 2-4-36. Level 2 plan view

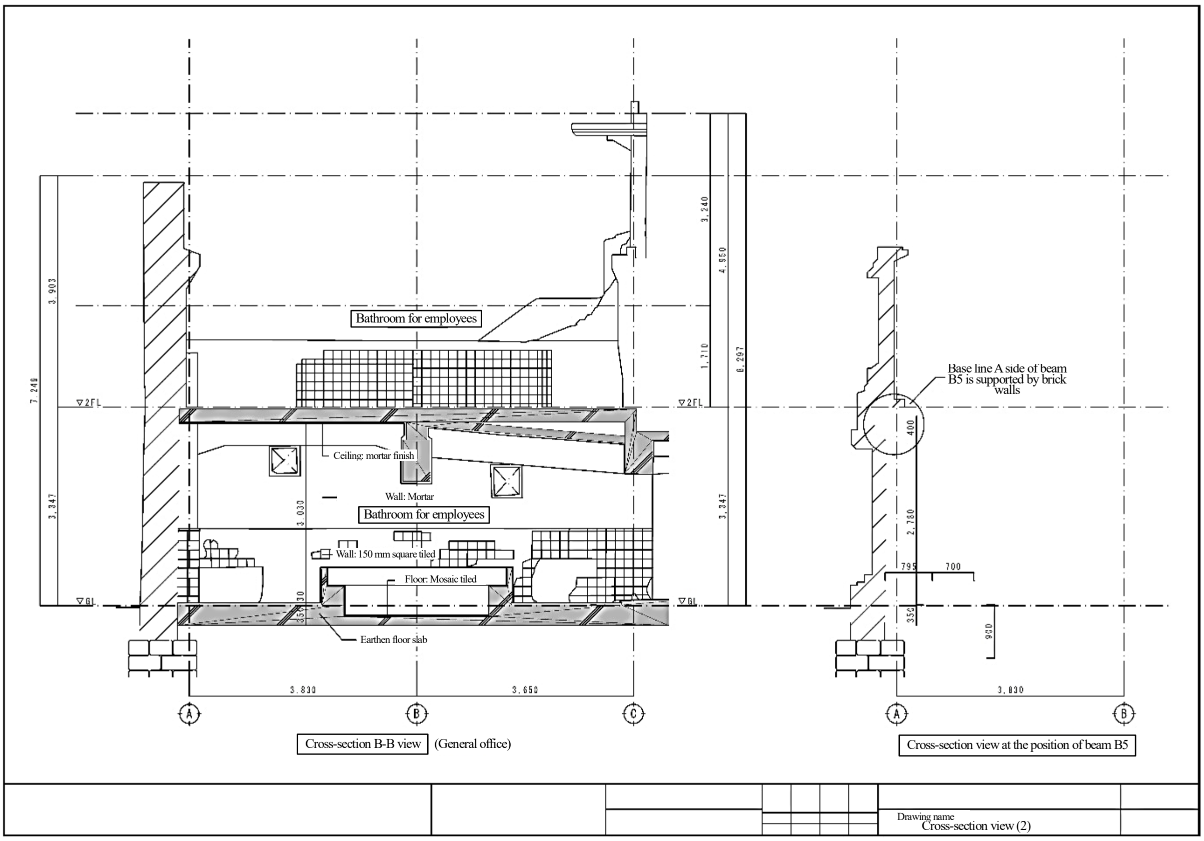


Figure 2-4-37. Cross-section view

Appendix 3

⑤ Surveys / tests

The following surveys and tests were conducted to collect data to understand the current conditions of the reinforced concrete:

⑤-1 Collection of samples and post-sampling restoration

- Core (⌀100): 4 locations
- Chippings (W200 × H200 × D40): 3 locations

* After the sampling, the sites were restored using non-shrink mortar.

⑤-2 Investigation of reinforced concrete for deterioration, etc.

- Visual examination of rebars: 3 sites (examination of thickness of concrete cover, rebar diameter, and degree of corrosion)
- Examination of bar arrangement: 26 sites (examination of locations and diameter of rebars using electromagnetic induction method)
- Compressive strength test: 4 bars
- Neutralization depth measurement: 7 bars
- Chloride ion content measurement: 12 slices
- Rebound hammer test: 26 sites
- Static elastic modulus test: 4 sites
- Reinforcement corrosion degree survey: 2 sites (self-potential measurement and polarization resistance measurement)
- Core drilling (to determine if there were foundations / underground beams): 2 sites

* After the sampling, the sites were restored using non-shrink mortar.

⑤-3 Microtremor measurement to observe the remains' characteristic period, etc.

Per the Common Specifications for the Site Surveys (established December 2011) (4.6.5 Microtremor measurement).

- Locations of measurement: 3 sites: upper, middle, and lower levels
- Directions of measurement: 3 components: horizontal (x and y axes) and vertical (z axis)
- Measurements: Microtremor (remains / ground) - 3 times; sandbag impact test (remains) - 1 time
- Measurement item: Acceleration
- Analyses: Predominant frequency of the ground and predominant frequency of the remains

⑤-4 Measurement of wind speed and direction, temperature, and humidity

- Locations of measurement: 3 m above the floor of Level 2
- Measurements: Zonal wind speed, meridional wind speed, vertical wind speed, and 10-minute average wind

speed

- Measurement period: Same with microtremor measurement

⑤-5 Survey of the foundations (exploratory drilling)

- Locations of exploratory drilling: 2 sites
  - ① Area of 1 m2 / depth of 0.9 m
  - ② Area of 1.6 m2 / depth of 0.78 m
- Objectives: Examining the conditions of the foundations (geometry, materials, etc.)
  Checking for any deterioration, subsidence, sloping, etc. of the foundations
  Determining whether or not there were underground pipework, and any damage to it at Site ②

⑥ Creation of structural drawings

Based on the results of the measurement, the visual examination of rebars and the Examination of bar arrangement, a series of structural drawings were created. A list of the drawings created is provided in Table 2-4-31. The Level 2 beams framing plan and the lists of column, girders, and beams were created based on the actual measurements taken of the cross-sectional dimensions of the members.

Table 2-4-31. List of drawings (structural drawings)

No	Name of drawing	Scale	Category	Remarks
01	Underground beams framing plan	1/50	Framing plan	S-01
02	Level 2 beams framing plan	1/50	Framing plan	S-02
03	Framing elevation (1)	1/100	Framing elevation	S-03
04	Framing elevation (2)	1/100	Framing elevation	S-04
05	Column / girder / beam / slab list	1/50	List	S-05

2) Outline of building surveyed

Name: General Office

Location: Hashima, Takashima-cho, Nagasaki City

Constructed in: 1896-1960

Structure: Reinforced concrete

Levels: 2

Current conditions: The building remains as part of a structure that contains brick walls of No. 3 winding machine room remains. The columns, beams, and the underside of the floor slab had the surface peeling off with rebars exposed, and the deterioration may progress even further. The roof and the outer walls of Level 2 have collapsed.

Surveyed building, outside (Eastern side)

Surveyed building, outside (Western side)

Surveyed building, inside (Bath)

Surveyed building, inside (Warehouse)

Photo 2-4-63. Appearance of General Office

3) Collection of samples and post-sampling restoration

① Objectives of survey

As a part of data collection to understand the current conditions of the reinforced concrete, concrete cores were drilled out as necessary for performing compressive strength test, static elastic modulus test, neutralization depth measurement, and chloride ion content measurement, and chipping was carried out as necessary for performing neutralization depth measurement, reinforcement corrosion degree survey, reinforcement corrosion survey, and visual check of the conditions and dimensions of rebars.

② Survey methods

Prior to core drilling and concrete chipping, rebar exploration was performed using the electromagnetic wave radar method. Based on the results of the rebar exploration, the bar arrangements were marked with tape for identification, before core drilling and concrete chipping were performed (Figure 2-4-38).

The diameter of a concrete core was determined based on the bar arrangements; it was drilled out at the diameter of 77 mm. The length of a core was to be from the starting point for drilling to the finished surface at the end (penetration) for the floors, and approximately twice the length of the diameter for beams, columns, and underground beams. The cores drilled out were wound with plastic wrap to protect them against neutralization before storage (Photo 2-4-64).

Chipping was performed where rebars ran orthogonal, within a range of 200 mm × 200 mm, and deep enough to determine the thickness of concrete cover for the rebars.

Rebar exploration

Core drilling work

Concrete chipping work

Curing of drilled cores

Photo 2-4-64. Core drilling and chipping work

Appendix 3

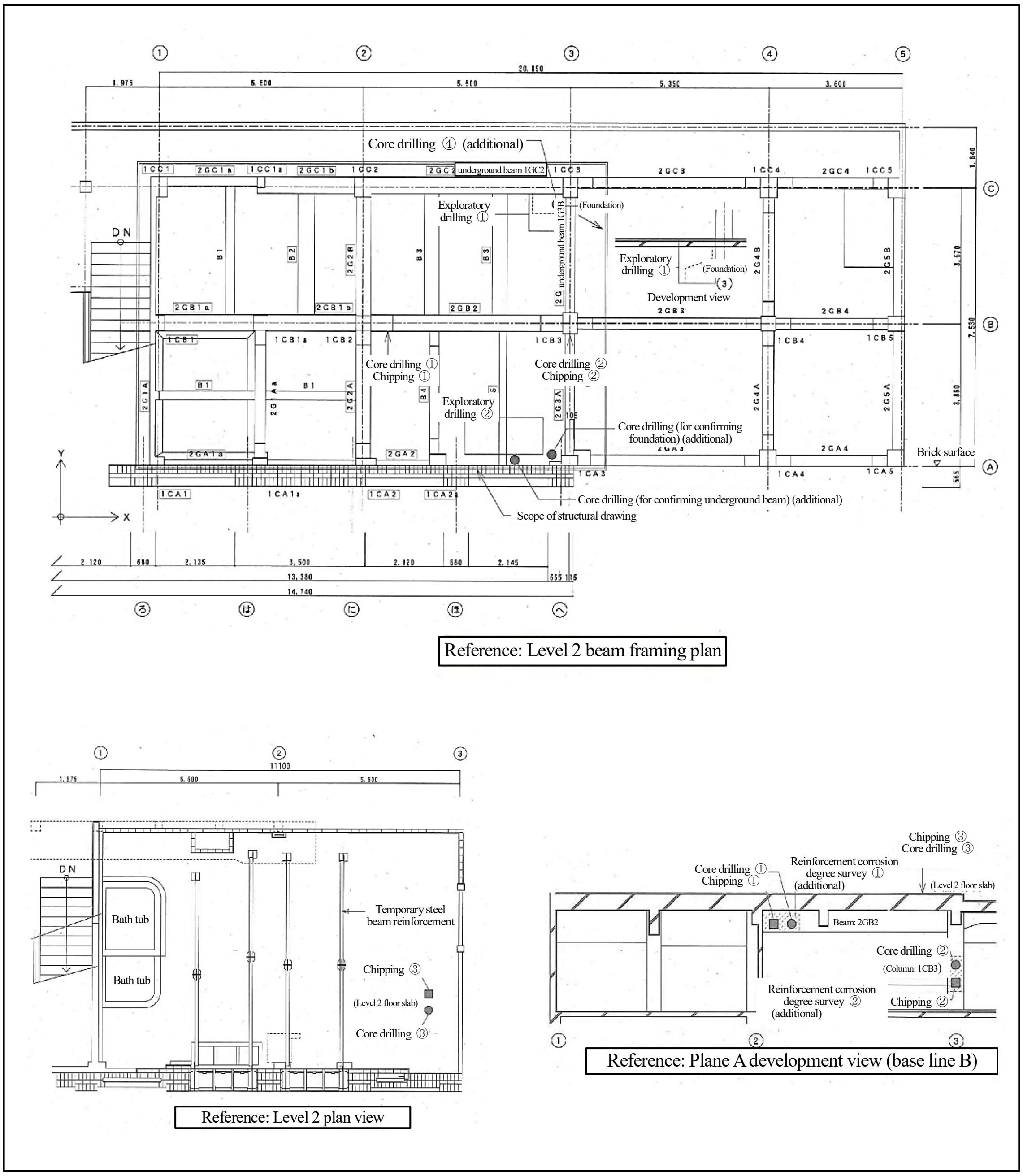


Figure 2-4-38. Core drilling / chipping locations

4) Investigation of reinforced concrete for deterioration, etc.

① Objectives of survey

A series of tests and surveys were performed on the cores collected and at the chipping sites to understand the current conditions of the reinforced concrete, for the purpose of preparing basic materials necessary in discussing future preservation and repair, as well as structural reinforcement. A list of the test / survey items and quantities is provided in Table 2-4-32, and diagrams of survey locations in Figures 2-4-39 and -40.

Table 2-4-32. Test / survey items and quantities

Item	Quantity	Summary
Visual examination of rebars	3 sites	Chipping Sites ①-③
Examination of bar arrangement (electromagnetic induction method)	34 sites	Areas shown in □
Compressive strength test	4 sites	Cores ①-④
Neutralization depth measurement	7 sites	Cores ①-④ and Chipping Sites ①-③
Chloride ion content measurement	12 slices	Cores ①-④ (3 slices for each core)
Rebound hammer test	34 sites	Areas shown in □
Static elastic modulus test	4 sites	Cores ①-④
Reinforcement corrosion survey (Self-potential measurement)	2 sites	Chipping Sites ① and ②
Reinforcement corrosion survey (Polarization resistance measurement)	2 sites	Chipping Sites ① and ②
Core drilling (to determine if there were foundations / underground beams)	2 sites	Cores marked with ●

Appendix 3

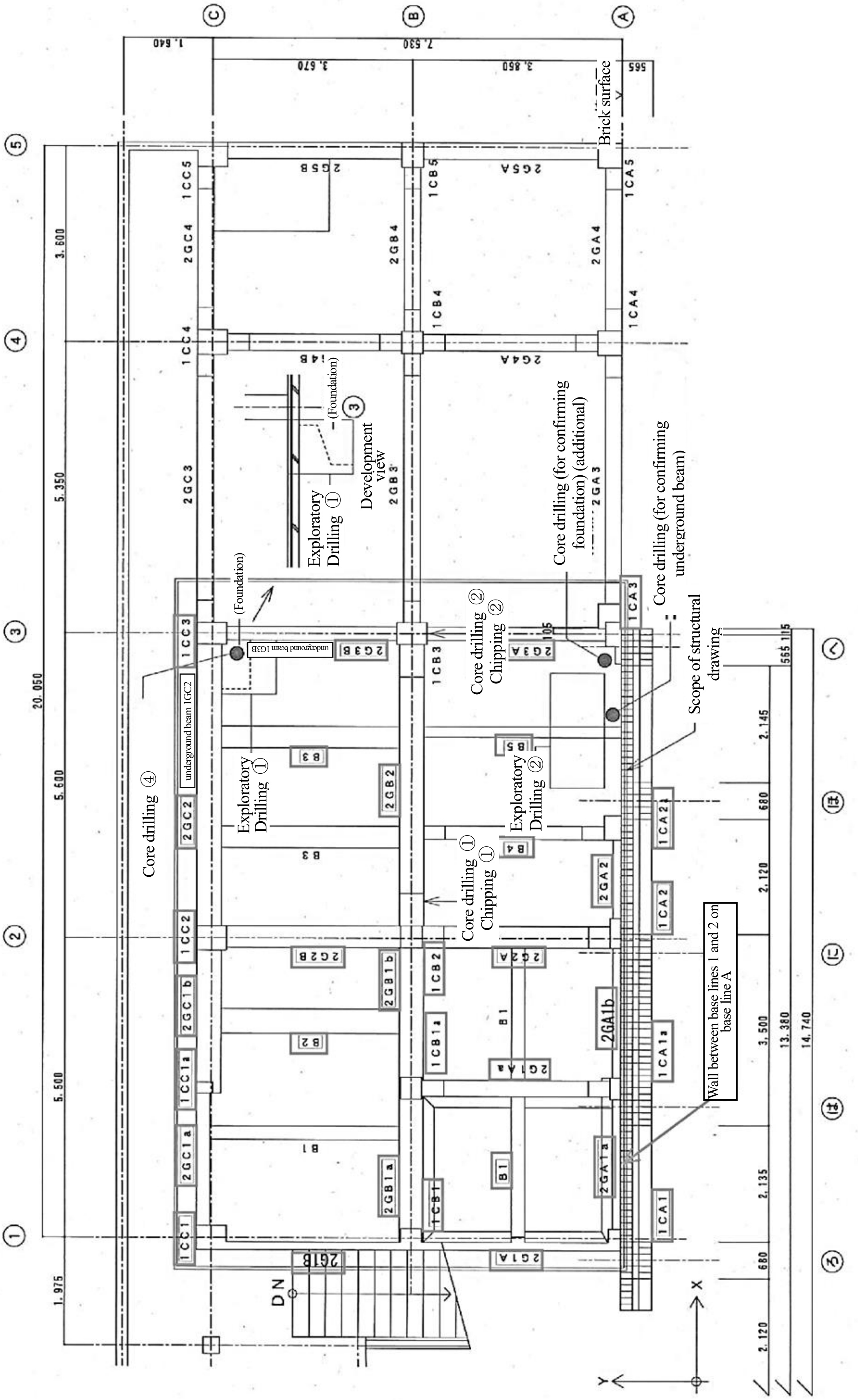


Figure 2-4-39. Test / survey locations (1)

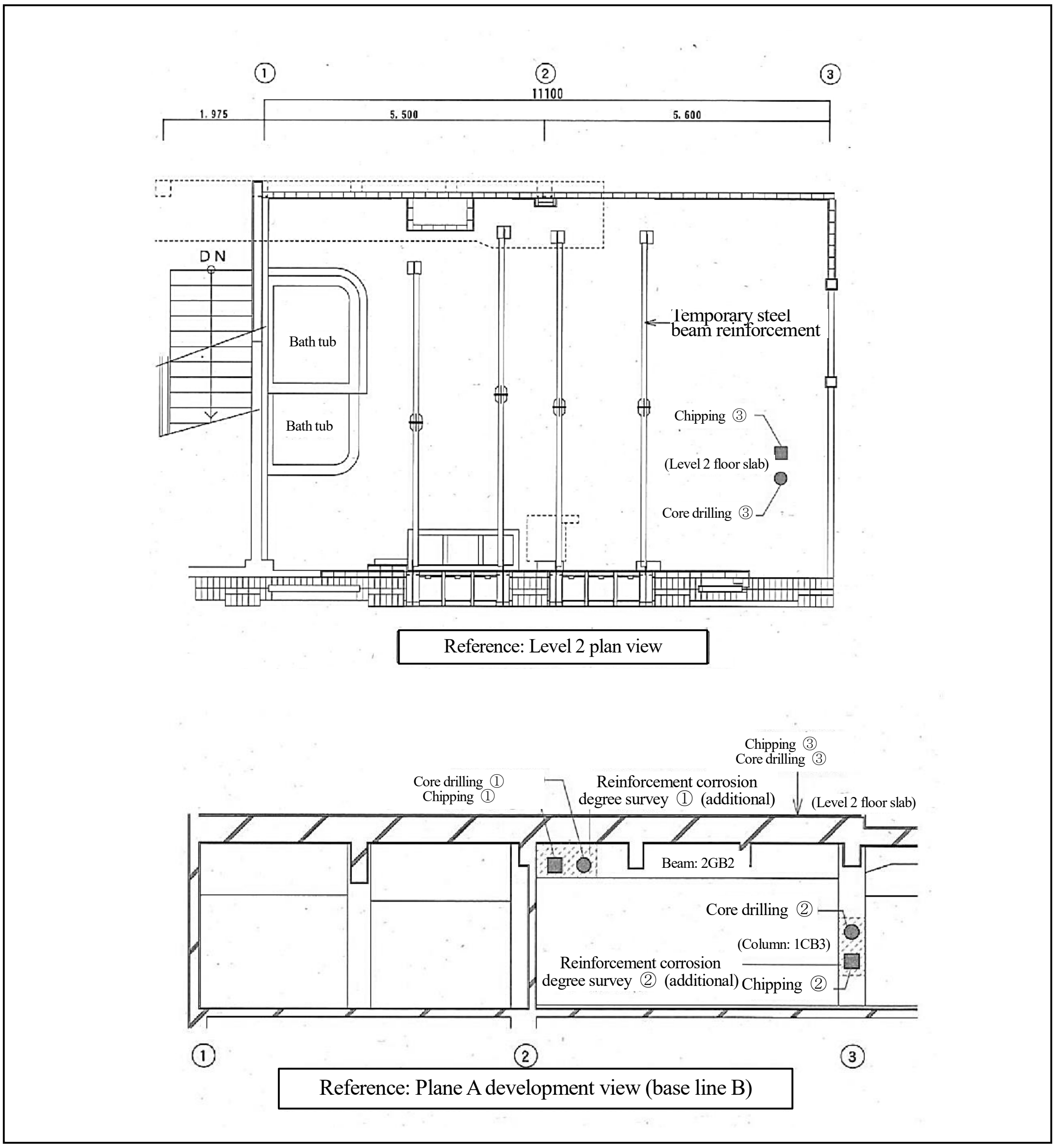


Figure 2-4-40. Test / survey locations (2)

② Visual examination of rebars

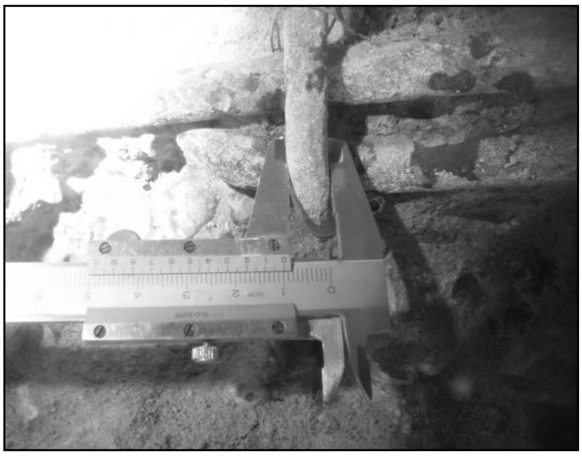
The examination was conducted in order to determine the bar arrangement, thickness of concrete cover, rebar diameter, and degree of rebar corrosion (at the chipping sites).

②-1 Survey methods

The rebar arrangements were examined using a rebar explorer (the electromagnetic wave radar method), and the bar arrangements were marked with tape for identification. Measurements were taken of the rebar spacing. The thickness of concrete cover for the rebars and rebar diameters were measured using scales and calipers at the chipping sites. The state of reinforcement corrosion was examined visually at the chipping sites, and graded according to the criteria for the levels of corrosion provided in *Concrete Diagnostic Techniques* (Japan Concrete Institute).

Rebar exploration (electromagnetic wave radar method) work

Rebar arrangement



Measuring rebar diameter

Measuring concrete cover thickness

Photo 2-4-65. Visual examination of rebars

②-2 Results

For the beams, columns, and floor, chipping was carried out to take measurements and examine the degree of corrosion visually. Reinforcement corrosion degree survey using self-potential measurement and polarization resistance measurement was also taken for the beams and columns, as well as measurements of the rebar diameter and concrete cover thickness using the electromagnetic induction method as a part of the Examination of bar arrangement.

The rebar diameter, rebar spacing, concrete cover thickness, and degree of rebar corrosion (as determined visually) measured at each measurement sites are provided in Table 2-4-33. The degree of rebar corrosion was Class III for the beams and columns, with floating rust observed around the circumference and along the entire length of the rebars. The rebars in the columns, in particular, had the hoops showing a loss of cross-sectional area, indicating significant levels of corrosion.

The thickness of concrete cover was greater at all the sites compared to the standards specified in the Order for Enforcement of the Building Standard Act (Photo 2-4-66).

A rebar exploration conducted on the beams from down below showed that there were only two rebars for the bottom reinforcement.

Rebar conditions (column)

Rebar conditions (beam)

Rebar conditions

Rebar arrangement at bottom of beams

Photo 2-4-66. Conditions of rebars

Appendix 3

Table 2-4-33. Summary of rebar survey results

No.	Site	Diameter of rebar		Rebar spacing (hoop and stirrup)	Concrete cover thickness	Level of corrosion
		Main reinforcement	Hoop (stirrup)			
①	Beam	13φ	9φ	150 mm	32 mm	III
②	Column	16φ	13φ	250 mm	43 mm	III
③	Floor	9φ	9φ	250 mm (along X axis) 200 mm (along Y axis)	53 mm	II

Table 2-4-34. Criteria for the levels of corrosion of rebar

Level of corrosion	Condition of rebar
I	Mill scale surface, or no rust formed or thin and fine layer of rust in general, and no rust on the concrete surface.
II	Spots of floating rust, but only in small speckles.
III	No loss of cross-sectional area observed visually, although floating rust observed around the circumference and along the entire length of the rebar.
IV	Loss of cross-sectional area

Table 2-4-35. List of concrete cover thickness specified in Order for Enforcement of the Building Standard Act

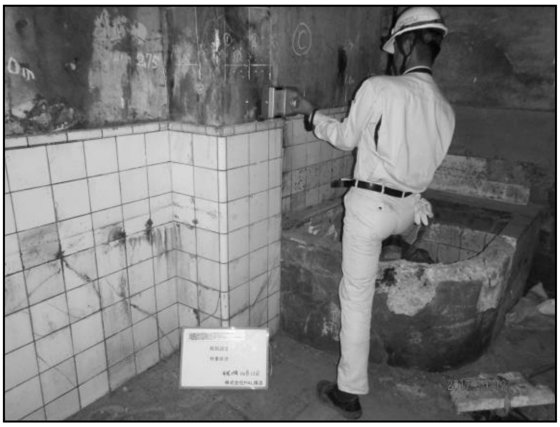
Areas			Values specified in the Enforcement Order
Areas not in contact with soil	Floor slabs / roof slabs Non-bearing walls	Indoor Outdoor	20 or greater
	Columns / beams / Bearing walls	Indoor Outdoor	30 or greater
	Retaining walls		-
Areas in contact with soil	Column / beam / floor slabs / roof slabs / walls Rising edge of continuous footing		40 or greater
	Foundations / retaining walls		60 or greater

③ Examination of bar arrangement

For the members for which chipping was not performed, non-destructive testing was conducted to determine the bar arrangements, diameters of rebars, and the thickness of concrete cover.

③-1 Testing methods

Rebar exploration was performed using the electromagnetic induction method at 34 sites of members for which chipping was not performed. A rebar explorer (with electromagnetic radars) was used to mark along the bar arrangements with chalk, while the electromagnetic induction method was used to measure the rebar diameter and concrete cover thickness.



Examining bar arrangement

Examining bar arrangement (2GC2, side)

Examining bar arrangement

Explorer display

Photo 2-4-67. Examination of bar arrangement

③-2 Results

Measurements were taken once along the x axis and another along the y axis for the columns, and on either side and the lower surface the of beams.

When the measurements taken of Chipping Site ① (beam) were compared to the diameter of rebar determined using the electromagnetic induction method, the measured diameter was 13 φ and 10 φ for the main reinforcement and the stirrup, respectively, while the diameter determined using the electromagnetic induction method was 16φ for both, namely the rebar diameters determined using the electromagnetic induction method were greater than those based on actual measurements. As similar results were produced in previous surveys conducted in Hashima, it was decided that the rebar diameter data based on the electromagnetic induction method should be used only as a guide. Considering that the structures in the areas of survey had been constructed in around the same period, that the chipping sites were of typical columns and beams representative of the building, and that the actual measurements were credible, the actual measurements of rebar diameters taken at the chipping sites were chosen to be used as estimates.

Table 2-4-36. Summary of rebar survey results

Site	ID	Rebar diameter (actual measurements and non-destructive testing)		Rebar diameter (estimate)		Rebar spacing (iron hoop and strap)	Concrete cover thickness	Remarks
		Rod	Iron hoop (strap)	Rod	Iron hoop (strap)			
Column	1CA1	16 φ	13 φ	16 φ	13 φ	250 mm	65 mm	
Column	1CA1a	25φ	16 φ	16 φ	13 φ	300 mm	66 mm	
Column	1CA2	16 φ	16 φ	16 φ	13 φ	250 mm	62 mm	
Column	1CA2a	19 φ	16 φ	16 φ	13 φ	300 mm	47 mm	
Column	1CA3	16 φ	16 φ	16 φ	13 φ	300 mm	82 mm	
Column	1CB1	16 φ	16 φ	16 φ	13 φ	250 mm	70 mm	
Column	1CB1a	19 φ	13 φ	16 φ	13 φ	200 mm	51 mm	
Column	1CB2	19 φ	16 φ	16 φ	13 φ	250 mm	63 mm	
Column	1CB3	**16 φ**	**13 φ**			250 mm	53 mm	Actual measurement
Column	1CC1	16 φ	10 φ	16 φ	13 φ	300 mm	35 mm	
Column	1CC1a	6 φ	6 φ	6 φ	6 φ	150 mm	120 mm (estimate)	Spiral reinforcement
Column	1CC2	19 φ	16 φ	16 φ	13 φ	300 mm	51 mm	
Column	1CC3	19 φ	13 φ	16 φ	13 φ	250 mm	38 mm	
Girder	2GA1a	19 φ	10 φ	13 φ	10 φ	200 mm	41 mm	
Girder	2GA1b	16 φ	16 φ	13 φ	10 φ	250 mm	53 mm	
Girder	2GA2	22 φ	16 φ	13 φ	10 φ	200 mm	79 mm	
Girder	2GB1a	19 φ	6 φ	13 φ	10 φ	350 mm	35 mm	
Girder	2GB1b	19 φ	6 φ	13 φ	10 φ	150 mm	37 mm	
Girder	2GB2	**13 φ** 16 φ	**10 φ** 16 φ			150 mm	44 mm	Actual measurement
Girder	2GC1a	22 φ	6 φ	13 φ	10 φ	250 mm	43 mm	
Girder	2GC1b	13 φ	10 φ	13 φ	10 φ	150 mm	38 mm	
Girder	2GC2	19 φ	13 φ	13 φ	10 φ	400 mm	84 mm	
Girder	2G1A	16 φ	13 φ	13 φ	10 φ	200 mm	59 mm	
Girder	2G1B	16 φ	6 φ	13 φ	10 φ	300 mm	48 mm	
Girder	2G1Aa	19 φ	16 φ	13 φ	10 φ	200 mm	53 mm	
Girder	2G2A	19 φ	13 φ	13 φ	10 φ	300 mm	55 mm	
Girder	2G2B	16 φ	16 φ	13 φ	10 φ	150 mm	83 mm	
Girder	2G3A	—	—	—	—	—	—	Unmeasurable due to rebar explosion
Girder	2G3B	16 φ	19 φ	13 φ	10 φ	150 mm	50 mm	
Beam	B1	16 φ	13 φ	13 φ	10 φ	200 mm	48 mm	
Beam	B2	16 φ	6 φ	13 φ	10 φ	150 mm	64 mm	
Beam	B3	16 φ	10 φ	13 φ	10 φ	150 mm	49 mm	
Beam	B4	19 φ	10 φ	13 φ	10 φ	250 mm	49 mm	
Beam	B5	13 φ	9 φ	13 φ	10 φ	200 mm	14 mm	
Floor	S1	**9 φ**	**9 φ**			250 mm (along X axis) 200 mm (along Y axis)	53 mm	Actual measurement

Figures in red represent actual measurements taken at a chipping site

④ Neutralization depth measurement

The depth of neutralization, which is a factor that deteriorates reinforced concrete, was measured to help understand the conditions that corrode the steel.

④-1 Testing methods

Measurements were taken at three points each of the drilled cores and the chipping sites. The test was performed by spraying a phenolphthalein solution 1% in ethanol over concrete, and areas taking on a reddish purple color was determined as not having been neutralized, and areas not taking on any color as having been neutralized. The depth of the latter was measured using a scale (Photo 2-4-68).

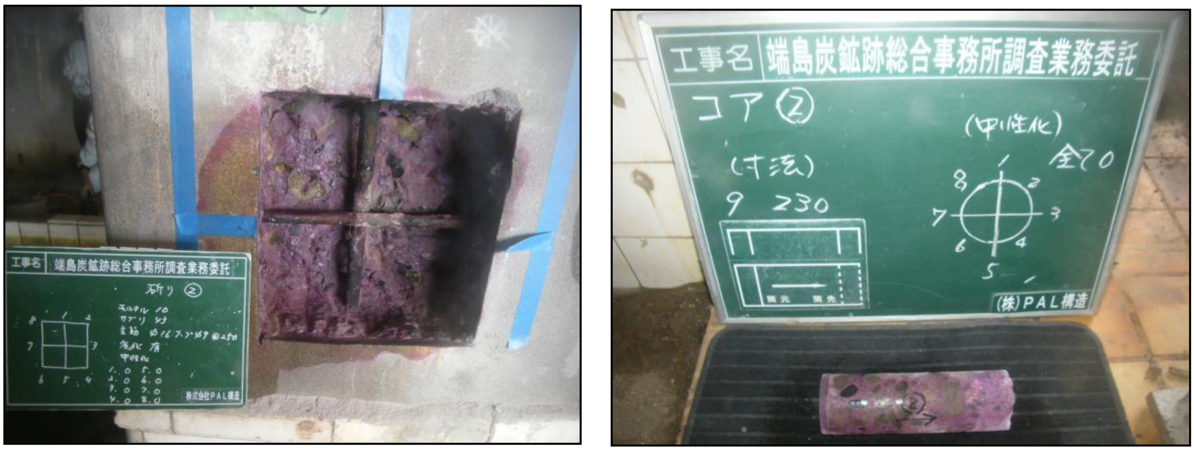


Photo 2-4-68. Testing for neutralization

④-2 Results

As shown in Table 2-4-37, the depth of neutralization was 0 mm for all measurement points, indicating that no neutralization had occurred. The results suggest that the corrosion of the rebars had not been caused by neutralization of concrete but by some other factors.

Table 2-4-37. Summary of neutralization depth measurement

No.	Site	Depth of neutralization (mean)		Thickness of concrete cover
		Core	Chipping site	
①	Beam	0 mm	0 mm	32 mm
②	Column	0 mm	0 mm	43 mm
③	Floor	0 mm	0 mm	53 mm
④	Foundation	0 mm	—	—

⑤ Chloride ion content measurement

The chloride ion content, which is a factor that deteriorates reinforced concrete, was measured to help understand the conditions that corrode the steel.

⑤-1 Testing methods

Test was performed on the drilled cores ①-④ by an officially certified institution (Aso Co.), according to the "Methods of test for chloride ion content in hardened concrete (JIS A 1154)." The drilled cores were cut at an interval of 2 cm, and the resulting slices were pulverized to measure the chloride ion content in concrete for each slice from different depth, using potentiometric titration. The mass of concrete per unit volume used for the calculation of chloride ion content was based on the apparent density determined by using the dimensions and mass measured in the compressive strength test. Three slices were cut from each core, and a total of 12 slices were tested (Photo 2-4-69).

Photo 2-4-69. Measurement of chloride ion content

⑤-2 Results

The results of this test showed that the chloride ion level exceeded the chloride threshold for corrosion in all of the cores on the surface, with the levels generally turning lower in deeper parts. In areas near rebars, the chloride ion levels exceeded the chloride threshold for corrosion in the columns and floor. The rebar corrosion levels were Class III for the columns and Class II for the floor, which show that the high salt levels near the rebars are likely to have been one of the causes of the rebar corrosion.

⑥ Compressive strength test and static elastic modulus test

The tests were conducted to determine the strength and deformation properties (i.e. the relationship between stress and distortion) of concrete in a direct manner to ascertain its soundness.

⑥-1 Testing methods

Tests were performed on the drilled cores ①-④ by an officially certified institution (Aso Co.), according to the

"Method of sampling and testing for compressive strength of drilled cores of concrete (JIS A 1107)" and the "Method of test for static modulus of elasticity of concrete (JIS A 1149)." Because the same samples were used for the chloride ion content measurement, they were not immersed in water, which is usually done to ensure the same wet-dry conditions.

For the cores ②-④, the test specimens' height-diameter ratio was smaller than 1.90; a correction factor was therefore used to calculate the compressive strength.

Photo 2-4-70. Performing compressive strength / static elastic modulus tests

⑥-2 Results

The test results are shown in Table 2-4-38. The compressive strength determined in the test ranged between 11.6 N/mm^2 and 42.4 N/mm^2. It was lower than the original design specification of 13.5 N/mm^2 for the beams. The static elastic modulus, meanwhile, was lower than the original design specifications for the columns, beams, and slabs. The decline in concrete strength may have been due to the blend of materials made at the time of construction, or deterioration with age.

Table 2-4-38. Summary of compressive strength / static elastic modulus tests

No.	Site	Mean diameter (mm)	Mean height (mm)	Maximum load (x 10^3N)	Correction factor	Compressive strength (N/mm^2)		Static elastic modulus (kN/mm^2)	Apparent density (g/cm^3)
						Before correction	After correction		
①	Beam	76.7	150.0	53.4	1.00	11.6	11.6	16.9	2.33
②	Column	76.7	143.8	89.8	0.99	19.4	19.2	11.8	2.21
③	Floor	76.7	134.1	200	0.98	43.3	42.4	16.3	2.36
④	Foundation	77.4	93.8	194	0.92	41.2	37.9	26.6	2.33

⑦ Rebound hammer test

Rebound hammer test was performed in order to determine in a simplified manner the strength of concrete from which cores had not been collected, and ascertain its soundness.

⑦-1 Testing methods

Rebound hammer test was performed at the same 34 sites where the examination of bar arrangement was conducted. A Schmidt hammer was used to make the surface of concrete flat and smooth, and being placed perpendicular to the surface where the measurements were taken, pressed slowly to produce impact.

Measurements were taken at 25 points for each site as a rule. Where it was difficult to take measurements at 25 points, the test was performed at 20 points at minimum.

Photo 2-4-71. Performing rebound hammer test

⑦-2 Results

The results from this test range between 18.3 N/mm^2 and 59.6 N/mm^2. When the data obtained from the compressive strength test were compared to those from the rebound hammer test for the core ① (Beam 2GB2), there was a large discrepancy was observed, with the former being 11.6 N/mm^2 compared to 59.3 N/mm^2 for the latter. Possible reasons for this discrepancy may include the unevenness of the finished surface and floating. The varying results in the estimated strength also suggest a possibility that some parts of the concrete may have lower strength than in other parts. This is suggested also by the results of the compressive strength test, as well.

⑧ Reinforcement corrosion degree survey

The survey was conducted to determine the likelihood of rebar corrosion and corrosion rates.

⑧-1 Survey methods

The survey was conducted on the Chipping Sites ① and ②, namely the column and beam which play important roles in structural performance of the building. The likelihood of corrosion was determined using self-potential measurement, and the corrosion rate by polarization resistance measurement (Photo 2-4-72).

Photo 2-4-72. Measuring reinforcement corrosion degree

⑧-2. Results

The assessment of corrosiveness and corrosion rate of rebars was performed according to the criteria provided in the *Concrete Diagnostic Techniques* (Japan Concrete Institute). With respect to the likelihood of corrosion, parts of the beam were judged to be “corroded with a probability of 90% or higher,” while others were “indeterminate” or “not corroded with a probability of 90% or higher.” According to visual inspection of the chipping sites, however, the level of corrosion was Class III for the column and beam, creating discrepancies in the assessment. Similarly, the corrosion rate as determined by the polarization resistance measurement also produced discrepancies with the results of “no corrosion” at all of the measurement points. A reason for these discrepancies may have been that even though the rust on the surface of the rebars had been polished off, corrosion had progressed deep inside, and taking measurements over the surface of the rust resulted in increased resistance.

⑨ Core drilling (to determine if there were foundations / underground beams)

At Exploratory Drilling Site ②, it was difficult to see foundations or underground beams by means of exploratory drilling. Core drilling was therefore carried out to determine if there were foundations or underground beams (Photo 2-4-73).

⑨-1 Survey methods

Rebar exploration was performed in the slab within the area of Exploratory Drilling Site ② to check for bar arrangements. The results found no rebar. Cores were collected by drilling straight into the slab in order to determine whether or not there were foundations or underground beams. The cores were drilled out at 100φ so that any layers could be recognizable on the surface.

Appendix 3

Drilling out a core

After a core has been drilled out

Cores drilled out

Restoration completed

Photo 2-4-73. Measuring reinforcement corrosion degree

⑨-2 Results

In this survey, a thin concrete foundation with a thickness of approximately 250 mm was observed. As a result of the core drilling and rebar exploration, however, it was confirmed that there was no underground beam there.

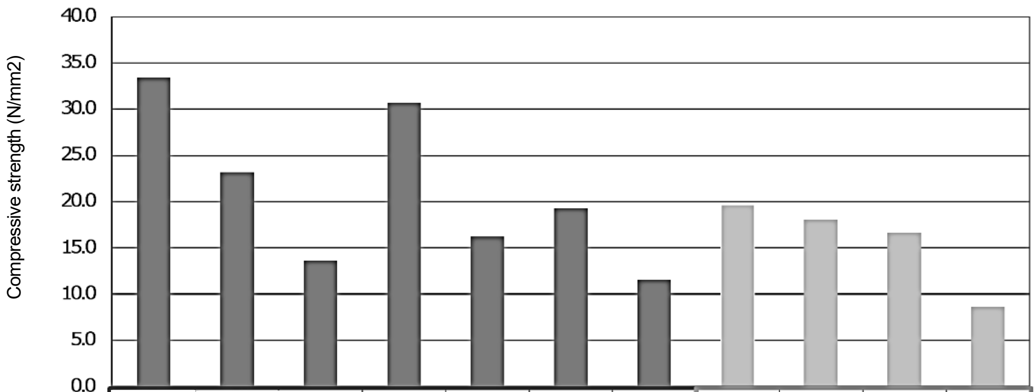
At both of the two core drilling sites, a layer of unreinforced concrete with a thickness of approximately 350 mm on the surface, and foundation stones were observed at about 600 mm below the surface. Considering that the concrete foundation was laid on the foundation stones and that no rebar had been found as a result of rebar exploration, it can be surmised that the foundation on the side of the brick walls was an unreinforced, eccentric foundation.

⑩ Discussion

The results of these tests and surveys showed that the compressive strength was lower than the original design specification of 13.5 N/mm^2 for the beams, and that the static elastic modulus, too, was lower than the original design specifications for the columns, beams, and slabs. The decline in concrete strength may have been due to the blend of materials made at the time of construction, or deterioration with age. The chloride ion content, meanwhile, was found to be higher at shallow levels. The chloride ion level exceeded the chloride threshold for corrosion where the rebars were, and rebar corrosion was also observed. Neutralization depth measurement, on the other hand, found little neutralization. Compared to other buildings on the island, this building had slightly lower levels of concrete strength.

Based on these findings about the current conditions of the General Office building, the concrete used in the building may have partial deterioration or decline in strength due to external factors or as a result of the blend of materials made at the time of construction. This also means that the corrosion of rebars may have partially progressed as a result.

Table 2-4-39. Compressive strength of buildings on Hashima Island



Bldg. 16, additional cover concrete (70 φ)	Bldg. 16, additional cover concrete (100 φ)	Bldg. 16, original	Bldg. 65	Bldg. 59	Bldg. 60	Bldg. 61	Bldg. 30, outside	Bldg. 16, outside	Bldg. 69	Bldg. 57
33.4	23.2	13.6	30.7	16.27	19.31	11.6	19.7	18.0	16.6	8.6
Cores collected by Gunkanjima WG							2005 survey (Surveys in association with a study on the technologies for the preservation and utilization of Gunkanjima)			

* Excerpt from "Report on the deterioration survey of concrete structures in Gunkanjima", Architectural Institute of Japan, et al.

5) Measurement of microtremor, wind speed and direction, temperature, and humidity

① Objectives of survey

As a part of the surveys to understand the current conditions of the remains of the General Office at Hashima Coal Mine, measurements of microtremor were taken for the purpose of obtaining data as a basis for the discussion of preservation, repair, and structural reinforcement.

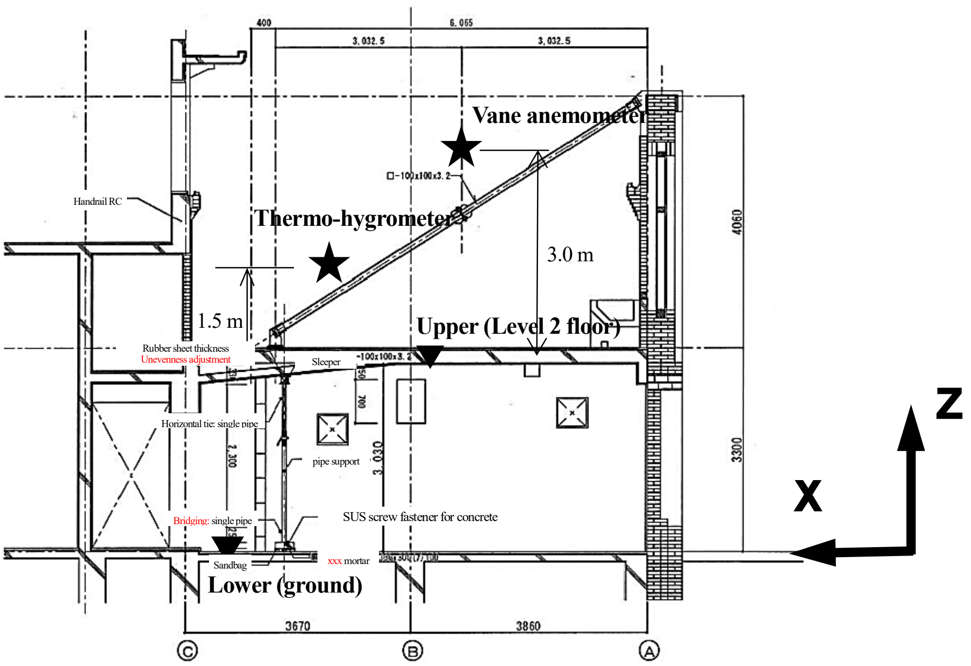
② Survey items and the quantities of samples

A list of measurement items and diagrams indicating the locations of measurement for the survey are provided in Table 2-4-40 and Figure 2-4-41, respectively. The survey took place from 8:00 until 17:00 on November 6, 2017.

Appendix 3

Table 2-4-40. Measurement items

Item		Locations of measurement	Measurements		Unit
Microtremor	Upper level	Floor, Level 2	3 components	X: Along shorter side of remains; Y: Along longer side of remains; Z: Vertical	Gal
	Middle level	Stairway	3 components	X: Along shorter side of remains; Y: Along longer side of remains; Z: Vertical	Gal
	Lower level	Ground	3 components	X: Along shorter side of remains; Y: Along longer side of remains; Z: Vertical	Gal
Wind speed and direction	Zonal (EW)	Level 2, 3.0 m from floor	Zonal wind speed during microtremor measurement (Eastward +)		m/s
	Meridional (NS)		Meridional wind speed during microtremor measurement (Northward +)		m/s
	Vertical (UD)		Vertical wind speed during microtremor measurement (Upward +)		m/s
	Average wind speed		10-minute average wind speed during microtremor measurement		m/s
Temperature and humidity	Temperature	Level 2, about 1.5 m from floor	External temperature during microtremor measurement		°C
	Humidity		Relative humidity during microtremor measurement		%RH



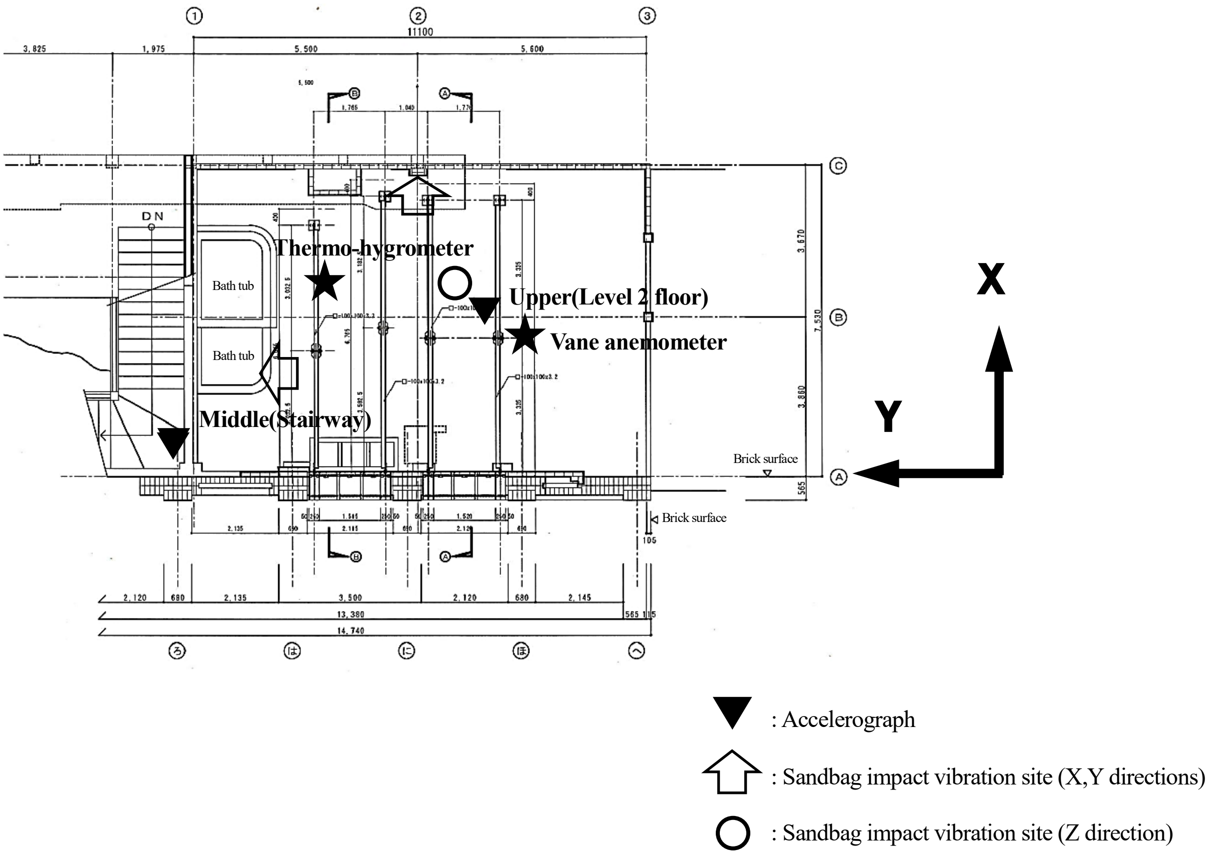


Figure 2-4-41. Locations of measurement

③ Measurements

At each measurement point, measurements were taken in 3 components, i.e. in horizontal (x and y axes) and vertical (z axis) directions, where the x direction is along the shorter sides of the remains and the y direction the longer side. The methods of measurement are shown in Table 2-4-41.

Table 2-4-41. Methods of measurement

Measurements	Measurement item	Sampling interval	Measurement time	Directions of measurement
Microtremor (remains / ground)	Acceleration	0.005 s (200 Hz)	600 s	x, y, and z directions
Sandbag impact vibration (remains)	Acceleration	0.005 s (200 Hz)	41 s	x, y, and z directions
Wind speed and direction	Wind direction / wind speed	1.0 s	Same with microtremor measurement	-
Temperature and humidity	Temperature / relative humidity	5 min	Same with microtremor measurement	-

④ Equipment

A list of pieces of equipment used and their key specifications are provided in Table 2-4-42.

Table 2-4-42. Equipment and specifications

Equipment	Model	Mfr.	Quantity	Key specs	Remarks
Servo accelerometer	LS-10C	Rion	9 (3 per measurement point)	1 ch / unit LS-10C in combination with LF-20 Frequency range: 0.02 - 100 Hz Acceleration range: ±3000 gal	—
Power-supply unit	FL-20	Rion	3 (3 per measurement point)		—
Data recorder	EDX-10B EDX-12A	Kyowa Electronic Instruments	1 3	3 x 4 ch / unit Resolution: 24bit	—
Process control PC	—	—	1	—	For controlling of data recording
Vane anemometer	SAT-600	Sonic	1	Ultrasonic Measurement range: 0 - 60 m/s	—
Thermo-hygrometer	HM-70	Vaisala	1	Temperature: -20 - 60°C Humidity: 0 - 100%RH	—
Miscellaneous: A set of equipment including extension cable dedicated to vibration meter, BNS cables to connect between devices, and connecting terminal block, etc.					

⑤ Measurement system

The system for the measurement of microtremor, wind speed and direction, temperature and humidity is shown in Figure 2-4-42. Readings from the accelerometer and the vane anemometer were recorded directly on the process control PC, while those from the thermo-hygrometer were first recorded to the memory of the device, then data were extracted from it.

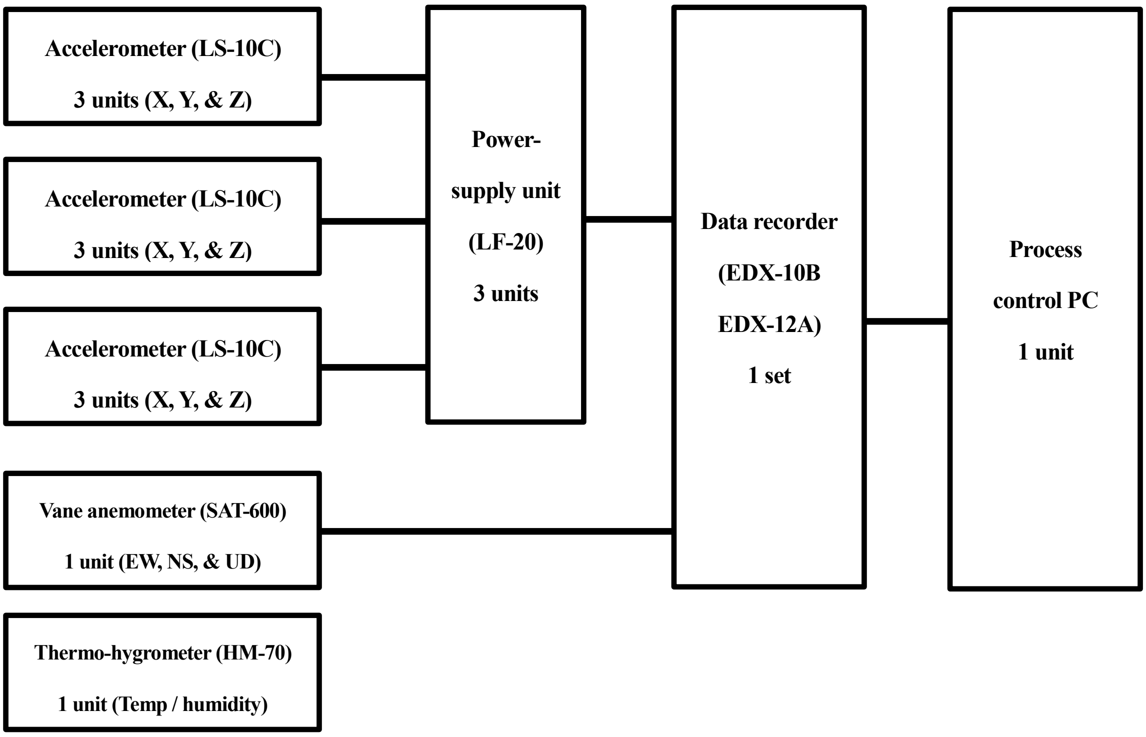


Figure 2-4-42. Block diagram of measurement system

⑥ Results

The results of the analysis are shown in Table 2-4-43.

Table 2-4-43. Recorded data

Microtremor ①	Mean wind speed: Approx. 3.5 m/s	Measurement started at 11:00
Microtremor ②	Mean wind speed: Approx. 3.0 m/s	Measurement started at 11:38
Microtremor ③	Little to no tremor	Measurement started at 13:27
Sandbag impact	A sandbag was let strike the surface from inside the building	Carried out between 12:34 and 12:49

Appendix 3

⑥-1 Results of microtremor measurements

The results of microtremor measurement (predominant frequency of the ground and the remains [Level 2 floor]) are shown in Table 2-4-44. The predominant frequency of the ground and of the remains was calculated using the H/V spectrum and the Fourier spectrum, respectively.

The calculations were performed in the manners described below.

○ Predominant frequency of the ground

1) Calculate the Fourier spectrum for different components (X, Y, and Z) from acceleration data of the ground.
2) Calculate the square-root of sum of squares of horizontal components (X, Y) of the Fourier spectrum.
3) Calculate the H/V spectrum by dividing the square-roots calculated in 2) by the Fourier spectrum of the Z-component.
4) Calculate the predominant frequency of the ground from the H/V spectrum calculated in 3).

○ Predominant frequency of the remains

1) Extract appropriate intervals from among the data for the Level 2 floor and stairways, and calculate Fourier spectrum by FFT.
2) Calculate the peak of the Fourier spectrum and the predominant frequency of the remains.

The results of the analyses are as follows:

Predominant frequency of the ground: Within the frequency range covered by the analysis, the H/V spectrum ratio 1.0 - 1.5, remaining largely constant with no clearly predominant frequency peak observed.

Predominant frequency of the remains: According to the results of the microtremor measurement and the sandbag impact test, the predominant frequency was 20.4 Hz (mean) for the X direction and 32.1 - 33.3 Hz (mean) for the Z direction. The values for the X and Z directions were reproducible, and were expected to represent the predominant frequency of the remains. For the Y direction, however, no clear peak or reproducibility was observed, and the predominant frequency was regarded as being unknown.

Table 2-4-44. Results of microtremor analysis

Methods of measurement				Predominant frequency (Hz)			
				Ground	Stairway	Level 2	
Microtremor		①		---	---	Z	32.0
		②		---	---	Z	29.5
		③		---	---	Z	34.8
Sandbag impact	Level 2 Wall / floor	X direction	1	---	---	X	20.1
			2	---	---	X	20.5
			3	---	---	X	20.6
		Y direction	1	---	---	Y	---
			2	---	---	Y	---
			3	---	---	Y	---
		Z direction	1	---	---	Z	33.4
			2	---	---	Z	33.3
			3	---	---	Z	33.2

⑥-2 Results of weather measurements (wind speed and direction, temperature, and humidity)

In parallel with the measurement of microtremor, measurements of wind speed and direction, temperature, and humidity were taken. The measurement period was the same as that for the microtremor measurement. The wind speed and direction were both plotted based on 10-minute averages. The results of the measurement are shown in Figures 2-4-43 and -44.

The results of weather measurement showed that the average wind speed peaked at around 10:10 at 4.6 m/s, then it became lighter into the afternoon, with the average speed falling below 1.0 m/s between around 12:30 and 13:40, indicating little to no wind. As for wind direction, it remained east-northeasterly in the morning, then turned largely southeasterly and southwesterly in the afternoon when the wind started becoming lighter.

Over the period of measurement of temperature and humidity, the average temperature was 20.3°C, and the average relative humidity was 62.9%RH. The lowest temperature was 17.0°C, from which it rose towards the midday, to the highs of 24.4°C. As the temperatures rose, the humidity gradually dropped, peaking at 82.3%RH first thing in the morning, down to the lowest at 42.2%RH. No violent change that might interfere with the microtremor measurement was observed either in the wind speed / direction, temperature, or humidity.

Appendix 3

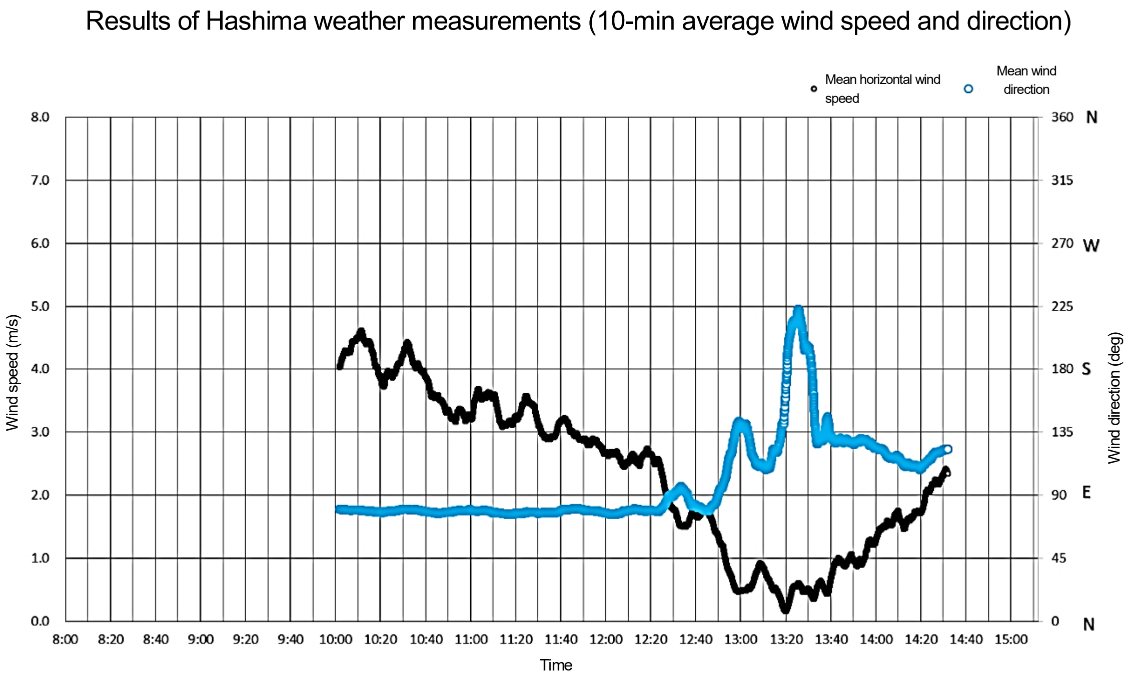


Figure 2-4-43. Measurements of wind speed and direction (10-minute average)

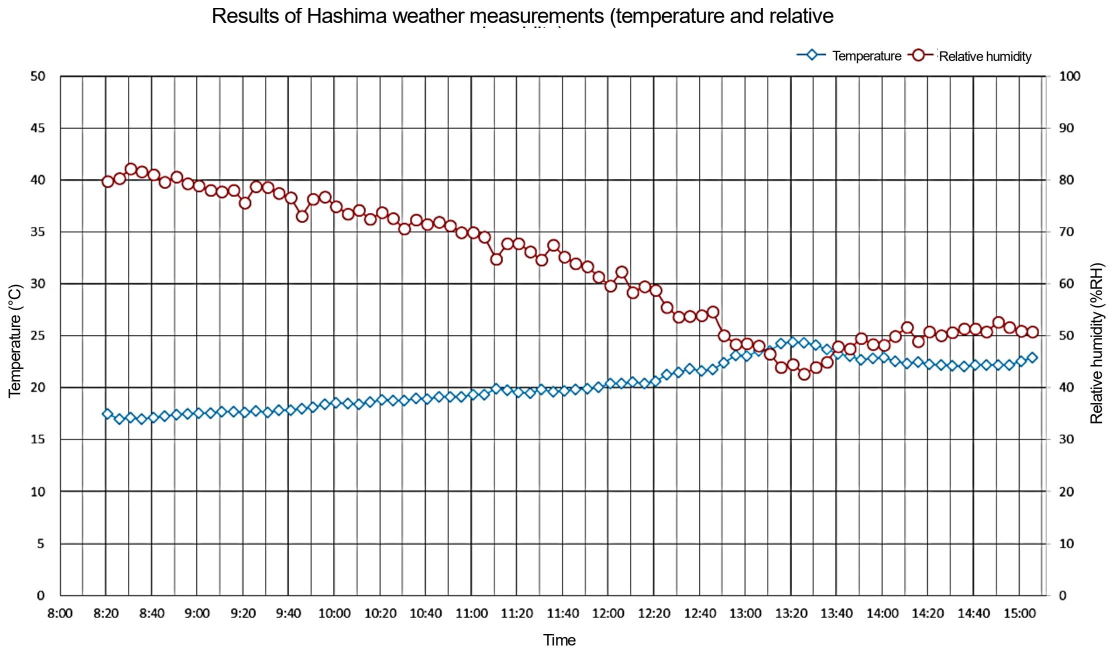


Figure 2-4-44. Measurements of temperature and humidity

⑦ Discussion

⑦-1 Predominant frequency of the remains

The General Office building covered in this survey had a relatively hard-to-vibrate structure compared to the remains surveyed in the past (FY2016: Brick walls of Pit No. 3 winding machine room; and FY2017: Mine entry landing), and the predominant frequency was determined only at two points: the X direction and the Z direction on Level 2. Predominant frequency could not be clearly determined in the Y direction, which ran along the longer sides of the remains, and the stairways.

Based on the predominant frequency determined, namely 20.4 Hz (mean) in the X direction on Level 2, and 32.1 - 33.3 Hz (mean) in the Z direction on Level 2 (floor vibration), it may be surmised that the building should not experience large resonance phenomena in the event of earthquake. However, if the rigidity of the building frame declines with the progress of deterioration at the remains, a possibility of the predominant frequency also becoming lower and thus inducing resonance phenomena cannot be ruled out.

As for the predominant frequency of the ground, the H/V spectrum ratio remained around 1.5 with no change (peak) in any frequency range, which suggests that the ground was relatively homogeneous with moderately firm soil.

⑦-2 Wind speed and direction, temperature, and humidity

No strong wind that might interfere with the microtremor measurement was observed.

6) Survey of foundations (exploratory drilling)

① Survey overview

A survey of the foundations was performing by means of exploratory drilling to examine the geometry and materials, and check for any deterioration, subsidence, etc., to understand the current conditions of the buried foundations of the remains, for the purpose of preparing basic materials necessary in discussing future preservation and repair, as well as structural reinforcement. An overview of the survey is provided below:

Date of survey: October 20, November 9, and November 21, 2017

Area and depth: Exploratory Drilling Site ① (Area: 1 m^2; depth: 0.9 m) / Exploratory Drilling Site ② (Area: 1.6 m^2; depth: 0.78 m)

Items unearthed: None

② Choice of sites of exploratory drilling

The object of the survey, the General Office building, is located in the southwestern part of Hashima Island, inside the area reclaimed in 1897. It is a reinforced concrete building, where Level 1 primarily housed a public bath for miners. For the purpose of examining the building foundations, the following sites were selected for exploratory drilling in the present survey: Exploratory Drilling Site ①, on the northern side of the bathtub in the

Appendix 3

west of the public bath; and Exploratory Drilling Site ②, on the southern side of the bathtub (Figure 2-4-45). The survey was performed by manually drilling out the floor, after the concrete laid over the floor had been removed. The diagrams of soil layers for Exploratory Drilling Sites ① and ② were created of the layers in the northern wall and in the southern wall, respectively. The reference altitude for the diagrams of soil layers was set at 8.934 m, or the altitude of the floor of the public bath.

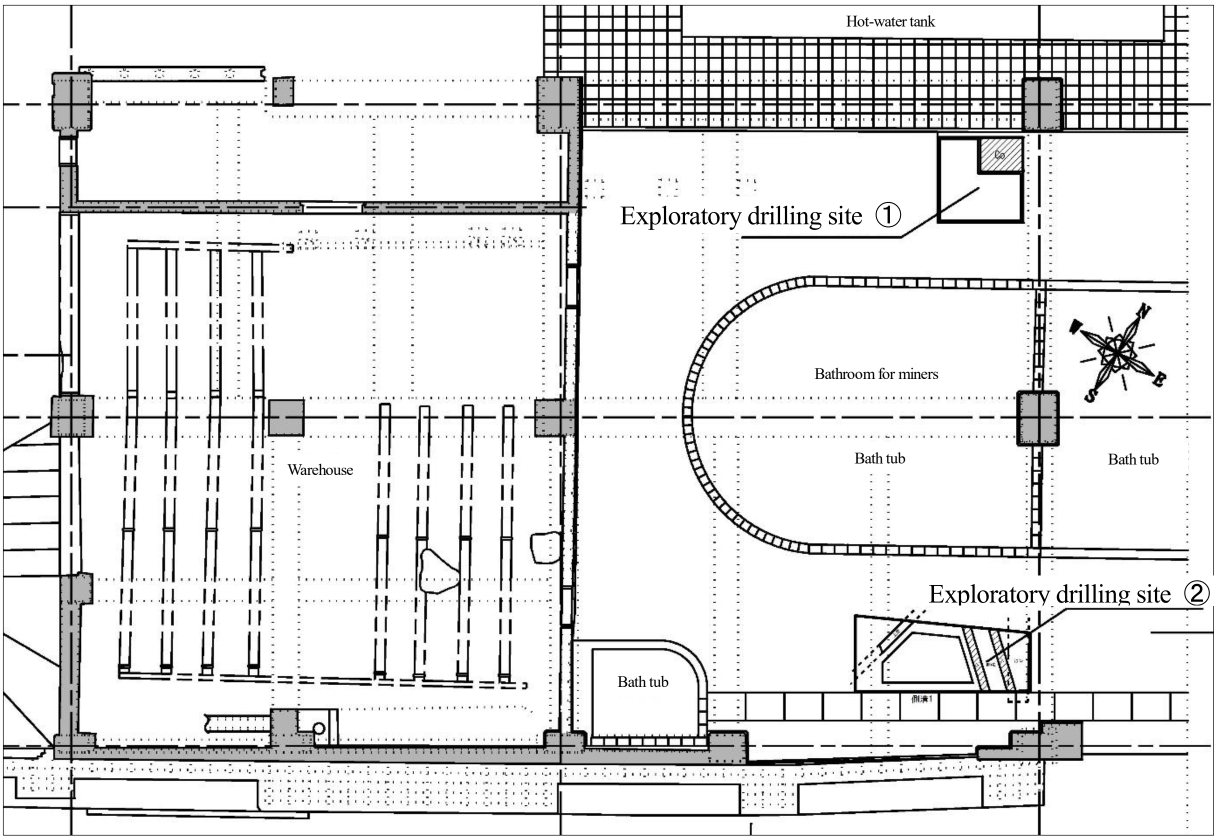


Figure 2-4-45. Locations of exploratory drilling sites

Photo 2-4-74. Exploratory Drilling Site ①, before start

Photo 2-4-75. Exploratory Drilling Site ①, survey underway

Photo 2-4-76. Exploratory Drilling Site ①, drilling completed

Photo 2-4-77. Exploratory Drilling Site ②, before start

Photo 2-4-78. Exploratory Drilling Site ②, survey underway

Photo 2-4-79. Exploratory Drilling Site ②, drilling completed

Photo 2-4-80. Exploratory Drilling Site ①, backfilling completed

Photo 2-4-81. Exploratory Drilling Site ②, backfilling completed

③ Exploratory Drilling Site ①

③-1 Base layers

A close inspection of the soil layers at Exploratory Drilling Site ① found a 35-cm concrete layer, laid over Layers 1 and 2 which are considered to have been filling soil layers (Table 2-4-45; Photo 2-4-82; Figure 2-4-47). No remnants were observed in either Layer 1 or 2. What is noteworthy is that the beginning of the sediment of filling soil layers (Level 1) was at nearly the same altitude (approx. 8.6 m above sea level) for both Exploratory Drilling Sites ① and ②. These sediment formations, combined with the fact the thickness of the concrete layer was approximately the same, appear to indicate that Exploratory Drilling Sites ① and ② were built in the same period.

Table 2-4-45. Exploratory Drilling Site ①, Summary of soil layers

Layer	Name	Soil color	Notes
1	Gray-brown sandy soil	Hue 2.5Y6/1 Yellowish brown	Sandy soil containing gravel of approx. 5 cm. Presumably laid for the purpose of filling the areas around the concrete foundation. No remnant found. Found at about the same altitude (approx. 8.6 m above sea level) as that for Layer 1 at Exploratory Drilling Site ②, though of a different soil type.
2	Blackish-brown soil with gravel	Hue7.5YR1/3 Blackish brown	Soil containing gravel and coal fragments of approx. 5 - 15 cm. Found beneath the concrete foundation, and considered to be filling soil , as with Layer 1.

Photo 2-4-82. Exploratory Drilling Site ①, Eastern wall

Photo 2-4-83. Exploratory Drilling Site ①, foundation

③-2 Exploratory drilling

Prior to manual drilling, a layer of concrete with a thickness of approximately 35 cm was removed from the floor, to reveal a concrete structure that was presumed to be the foundation (Photos 2-4-84 and -85). When the areas around the foundation were drilled out, the undersurface was observed at 40 cm deep. The depth from the floor level to the undersurface of the foundation was 75 cm. At Exploratory Drilling Site ①, the existence of a concrete structure that was presumed to be a foundation of the building was confirmed. No deterioration or subsidence was observed with the foundation.

Photo 2-4-84. Exploratory Drilling Site ①, drilling completed

Photo 2-4-85 Exploratory Drilling Site ①, foundation

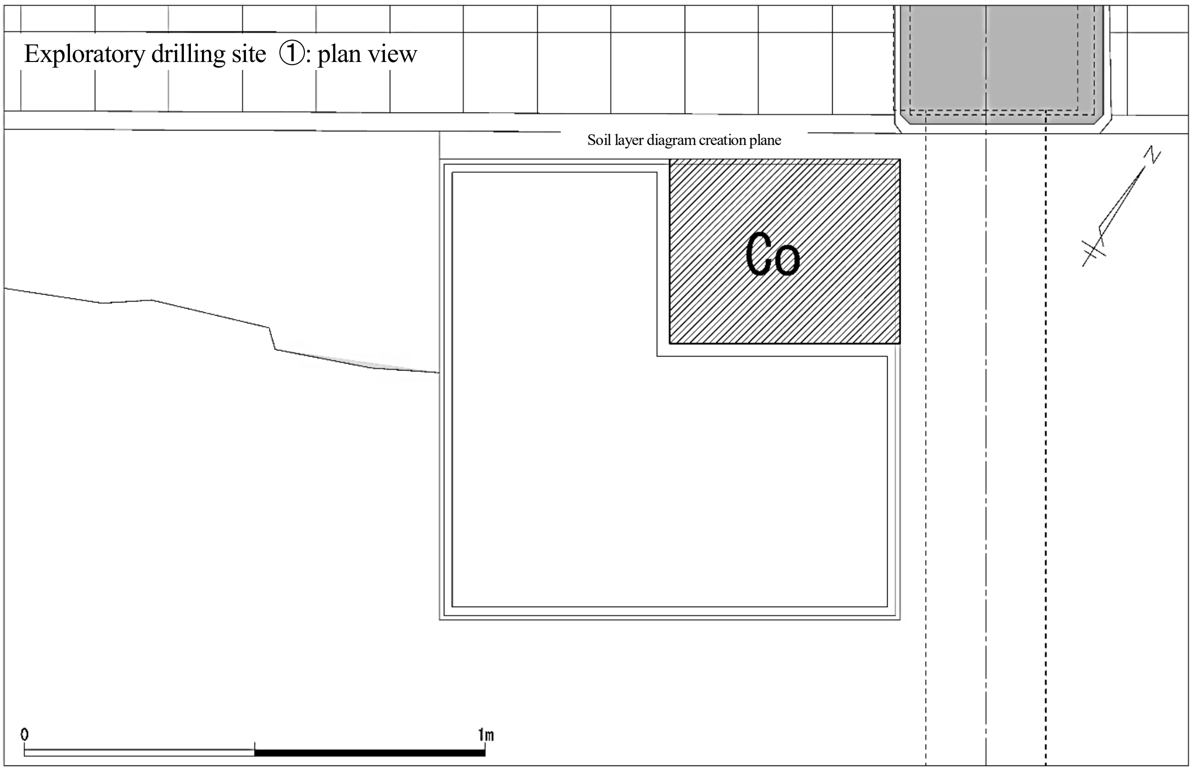
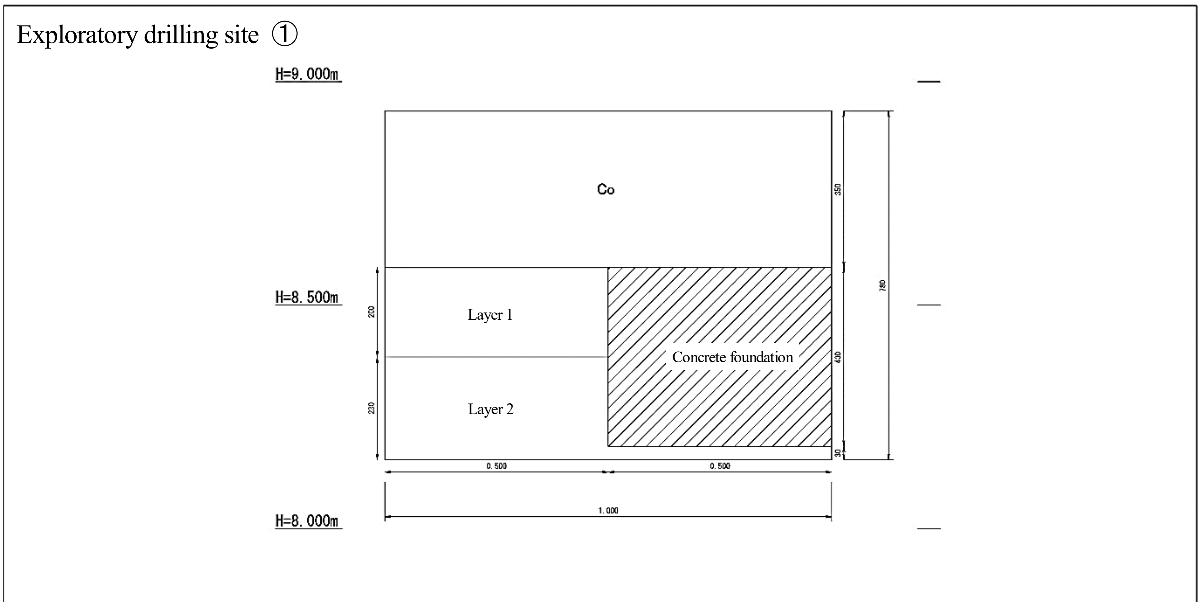


Figure 2-4-46. Exploratory Drilling Site ①, plan view



[Notes on soil layers]
Layer 1: Gray-brown sandy soil (Hue 2.5Y6/1 Yellowish brown)
Sandy soil containing gravel of approx. 5 cm. Presumably laid for the purpose of filling the areas around the concrete foundation. No remnant found. Found at about the same altitude (approx. 8.6 m above sea level) as that for Layer 1 at Exploratory Drilling Site ②, though of a different soil type.
Layer 2: Blackish-brown soil with gravel (Hue7.5YR1/3 Blackish brown)
Soil containing gravel and coal fragments of approx. 5–15 cm. Found beneath the concrete foundation, and considered to be filling soil, as with Layer 1.

Figure 2-4-47. Exploratory Drilling Site ①, diagram of soil layers in Northern wall

Appendix 3

④ Exploratory Drilling Site ②

④-1 Base layers

A close inspection of the soil layers at Exploratory Drilling Site ② found a 5-cm concrete layer, beneath which was a layer of crushed stone with a thickness of approximately 5 cm, followed by another layer of concrete and crushed stone with a thickness of approximately 25 cm. Underneath these concrete layers with the combined thickness of approximately 35 cm was a 20-cm filling soil layer, or Layer 1, under which was a layer of a traditional earth mixture, presumably Amakawa, with a thickness of approximately 5 cm (Layer 2). Underneath the Amakawa layer was a deposition of sandy soil (Layer 3) (Photos 2-4-86 and -87). No remnant was found in either Layers 1, 2, or 3.

Table 2-4-46. Exploratory Drilling Site②, Summary of soil layers

Layer	Name	Soil color	Notes
1	Dark brown soil with gravel	Hue 7.5YR3/3 Dark brown	Sedimentary soil found above Layer 2 (Amakawa soil), containing small gravel and concrete and coal fragments of approx. 5 cm. Presumably placed during the construction of the building. No remnant found. Found at about the same altitude (approx. 8.6 m above sea level) as that for Layer 1 at Exploratory Drilling Site ①, though of a different soil type.
2	Dark reddish-brown stiff soil	Hue 5YR3/4 Dark reddish brown	Very stiff, compacted red soil. Containing red soil as well as plaster; considered a type of traditional earth mixture ("Amakawa"). Presumably placed in the process of land reclamation and gutter installation, for the purpose of packing the ground and protecting against water infiltration from the lower layer.
3	Dark brown sandy soil with gravel	Hue 5YR4/2 Gray brown	Sedimentary soil found under Layer 2 (Amakawa soil). Sandy soil containing gravel of about 10 - 15 cm. The lower it gets, the wetter it becomes. It is unknown if it was soil brought in to fill the areas around the General Office, or it was sedimentary soil that had naturally deposited on the rock reef. Being sandy soil, it may also have been soil that had deposited in the area prior to the land reclamation. No remnant found.

Photo 2-4-86. Exploratory Drilling Site②, Southern wall

Photo 2-4-87. Exploratory Drilling Site②, Southern wall

④-2 Exploratory drilling

Exploratory Drilling Site ② was situated along the gutter (Gutter 1) on the southern side of the building (Photo 2-4-88). Prior to manual drilling, a layer of concrete was removed from the floor, to reveal a gutter that ran northwestward (Gutter 2) (Photos 2-4-88 and -89). When a layer of crushed stones of approximately 5 cm in thickness that had deposited underneath the concrete was removed, a surface of what appeared to have been a concrete floor slab was observed (Photo 2-4-88). The concrete floor slab was cut off diagonally on the western side, suggesting a possibility of it being one side of a gutter. Details are unknown, however.

Underneath the concrete floor slab was a layer of crushed stones, and together they were approximately 25 cm thick. The level below the concrete floor slab was Layer 1, which consisted of filling soil. Beneath this was a layer of a traditional earth mixture, presumably Amakawa. On the eastern side, an iron pipe with a diameter of approximately 23 cm was found at 20 cm below the ground level (Photos 2-4-89 and -90). The iron pipe was laid so that it ran through Gutter 2 and Gutter 1 (Photo 2-4-90). As the iron pipe and Gutter 2 both led towards the bathtub, it can be surmised that they were used to drain water in the bathtub. As for in what order the gutter and the pipe were built, it was either “the iron pipe was laid first, then the gutter was installed so that it overlapped the pipe,” or “the gutter and the pipe were laid at the same time,” considering it is difficult to lay an iron pipe through a concrete gutter. Whatever the case may have been, it would be difficult to create such a formation as this using a standardized gutter, it is likely that Gutters 1 and 2 were formed on site. The thickness of the bottom was approximately 7 cm. Meanwhile, a soil pipe with a diameter of approximately 10 cm was observed from directly above Layer 2 on the eastern side. While it ran in the opposite direction from Gutters 1 and 2, it is possible that it may also have been used for bathtub drain (Photo 2-4-90).

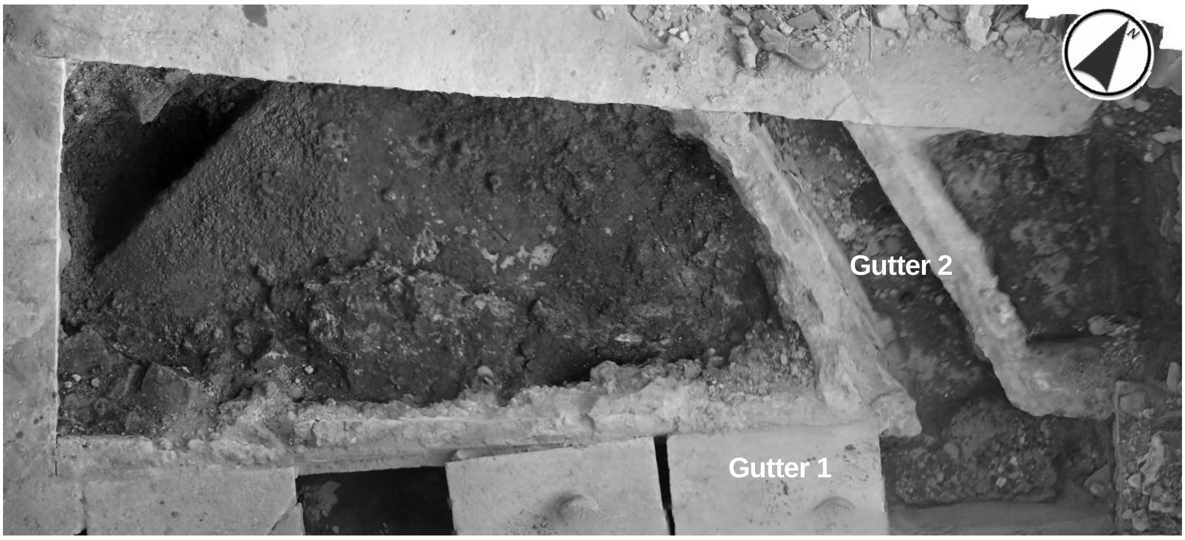


Photo 2-4-88. Exploratory Drilling Site ②, viewed from south

Photo 2-4-89. Exploratory Drilling Site ②, examination of iron pipe and Layer 2, viewed from south

Photo 2-4-90. Close-up (iron pipe)

Photo 2-4-91. Exploratory Drilling Site ②, side of Gutter 1 and Layer 2, viewed from north

Photo 2-4-92 Exploratory Drilling Site ②, plane view of Layer 2 and soil pipe

While the depth of the gutters was approximately 55 cm and 50 cm for Gutter 1 and Gutter 2, respectively, the undersurface of the gutters was almost same at around 8.4 m above sea level for both of them, with a layer of Amakawa laid at the level of the undersurface (Photo 2-4-94). While the reason why a layer of Amakawa was laid at the level of the undersurface of the gutters is unknown, it may have been done for the purpose of packing the ground and cementing the gutters during land reclamation, as well as to shut out the moisture coming from the lower layer (Layer 3) and protect the lower layer against infiltration with water that might leak from the gutters. Beneath Layer 2, or the Amakawa layer, was Layer 3, which presumably consisted of soil fills. Layer 3 was sandy soil with gravel, and was wetter in lower parts. Considering that it was sandy soil and that it was wet, it is possible that Layer 3 contained soil that had deposited there from before the land reclamation.

Photo 2-4-93. Exploratory Drilling Site ②, drilling completed; viewed from south

Photo 2-4-94 Exploratory Drilling Site ②, Southern wall

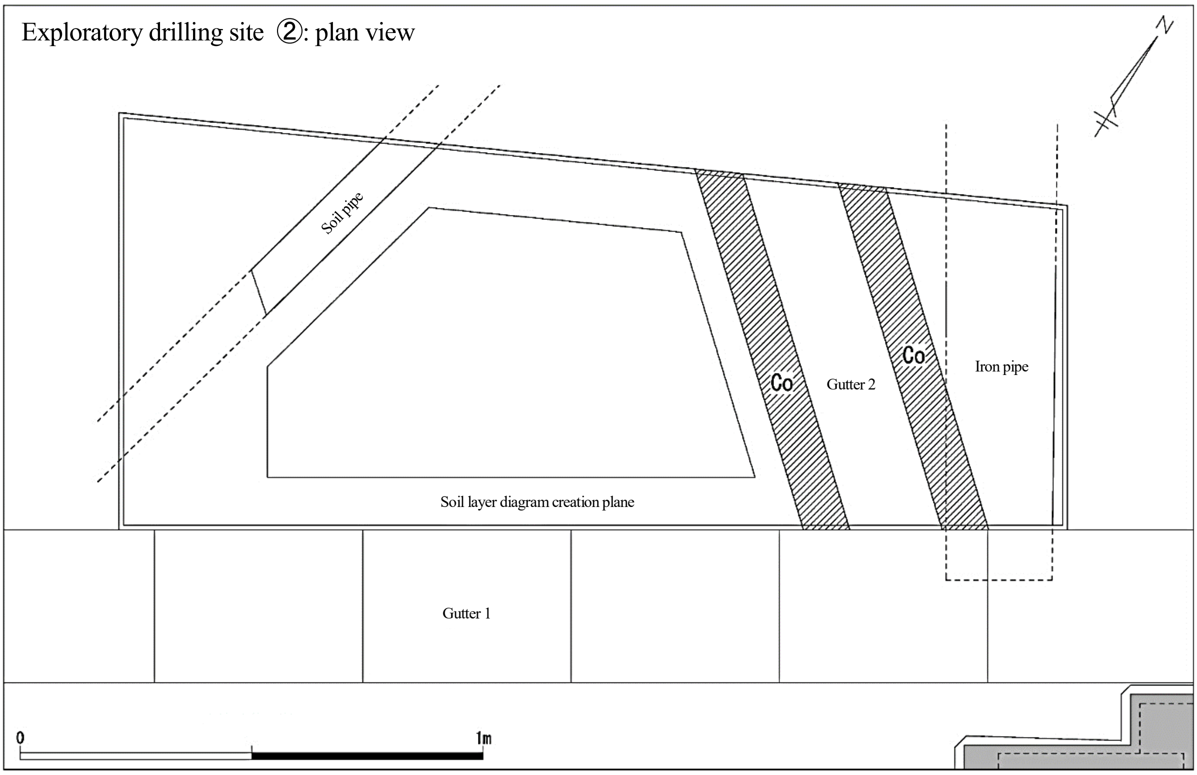
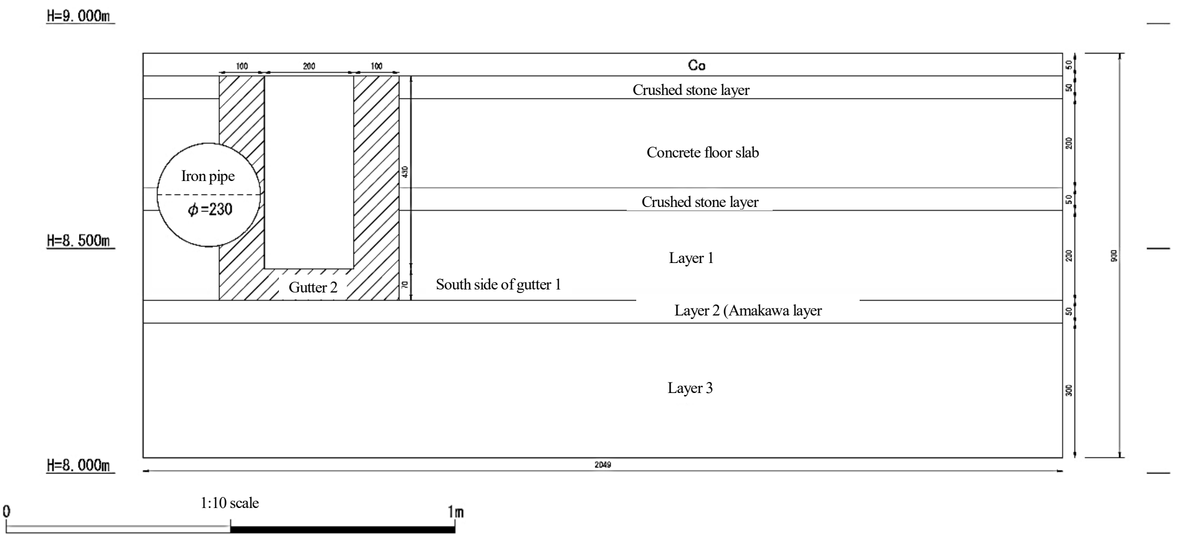


Figure 2-4-48. Exploratory Drilling Site ②, plan view

Exploratory drilling site ②



[Notes on soil layers]

Layer 1: Dark brown soil with gravel (Hue 7.5YR3/3 Dark brown)

Sedimentary soil found above Layer 2 (Amakawa soil), containing small gravel and concrete and coal fragments of approx. 5 cm. Presumably placed during the construction of the building. No remnant found. Found at about the same altitude (approx. 8.6 m above sea level) as that for Layer 1 at Exploratory Drilling Site ①, though of a different soil type.

Layer 2: Dark reddish-brown stiff soil (Hue 5YR3/4 Dark reddish brown)

Very stiff, compacted red soil. Containing red soil as well as plaster; considered a type of traditional earth mixture ("Amakawa"). Presumably placed in the process of land reclamation and gutter installation, for the purpose of packing the ground and protecting against water infiltration from the lower layer.

Layer 3: Dark brown sandy soil with gravel (Hue5YR4/2 Gray brown)

Sedimentary soil found under Layer 2 (Amakawa soil). Sandy soil containing gravel of 10 - 15 cm. Wetter in lower parts. It is unknown if it was soil brought in to fill the areas around the General Office, or it was sedimentary soil that had naturally deposited on the rock reef. Being

Figure 2-4-49. Exploratory Drilling Site ②, diagram of soil layers in Southern wall

⑤ Summary

As a result of the exploratory drilling, the foundation of the building could be observed at Exploratory Drilling Site ①, but not at Exploratory Drilling Site ②, where it was necessary to preserve the gutters and drilling could not be continued further. However, the existence of the foundation was confirmed at Exploratory Drilling Site ② by means of core drilling. At Exploratory Drilling Site ①, the foundation showed no damage, and no evidence of subsidence was found, although it was based only on visual examination. The rebar exploration found that foundation to be unreinforced. No underground beam was found at either Exploratory Drilling Site ① or ②. In addition to the findings mentioned above, Exploratory Drilling Site ② was found to have underground structures including gutters, iron pipe, soil pipe, and earth mixture preserved in favorable condition. This has provided us with invaluable information that helps us to understand the structures of the building.

7) Measurement of land subsidence

Considering the land subsidence observed on the eastern side of the brick walls, a slip were placed for the purpose of checking the altitude at the front of the brick walls, so that the data should serve as a reference for any change in the amount of subsidence over time.

⑦-1 Survey methods

An automatic level was used to measure the altitude at four points where a slip was placed in front of the brick walls, and at three points on top of the bricks. For the purpose of measurement, Class III reference mark No. 4 (10.652 m above sea level), which is placed along the southern coast of Hashima Island, was used. In case of a loss of reference marks due to waves or the collapse of seawall, a supplementary reference mark was established further inland at TP (Tokyo Peil, 8.877 m above sea level).

⑦-2 Results

Date of measurement: December 7, 2015

Level measurements	
Measurement point	Altitude
№1	8.883
№2	8.874
№3	8.889
№4	8.830
№5	9.286
№6	9.302
№7	9.232

8) Discussion

① Current conditions

①-1 Conditions of damage

General Office

Mortar had fallen off, or was floating, in many parts in the following areas: the outer walls and the eaves on Level 2, the inner walls and the underside of the slabs on Level 2 of the warehouse, and some of the columns and beams on Level 1. A possible cause for the mortar to float may have been mortar shrinkage due to changes in temperatures, allowing water to infiltrate.

Parts of the southeast side of the inner wall of plane C (South) (Photo 2-4-95), which is situated on the back side of the brick walls, were not reinforced concrete wall but had mortar laid directly on the back side of the brick walls. There were cracks in the mortar finish along the cracks in Arch No. 4 of the brick walls, in addition to mortar floating or fallen off in other parts.

Cracks were observed in the entire area. While many were smaller than 1.0 mm in width, some cracks were wider than 5.00 mm or grown almost causing damage. Hexagonal pattern cracks in mortar such as those found in plane A (North) (Photo 2-4-96) are considered to have been caused by the mortar expanding and shrinking due to changes in temperature. Hexagonal pattern cracks such as these were found primarily in the column, beams, and walls of the warehouse on Level 1, and the slabs on Level 2. Cracks running vertically along the columns, such as those found in the Column on the intersection between base lines C and 2, Level 1 (Photo 2-4-97), occurred along the main reinforcements and are considered to have been caused as a result of the expansion of the main reinforcements. Other cracks were presumably caused by mortar shrinkage due to changes in temperature.

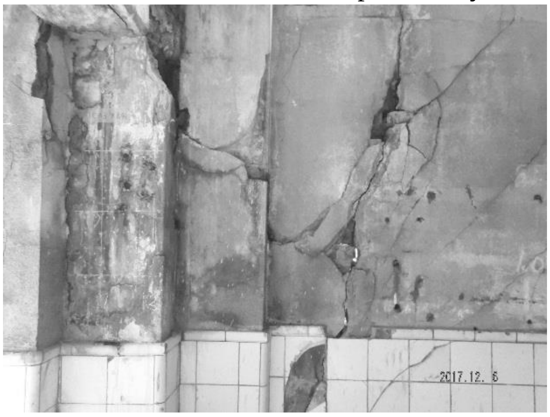


Photo 2-4-95. Mortar cracks and falling-off; plane C (South), inner wall

Photo 2-4-96. Plane A (North), outer wall

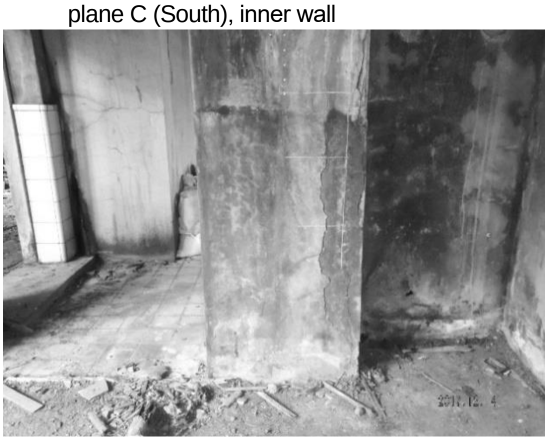


Photo 2-4-97. Cracks; column on base lines C and 2 intersection, Level 1

Some of the columns on Level 1 and beams on Level 2 suffered explosion in several parts, including those where rebars inside corroded following explosion, as seen on base lines A and 2 intersection, Level 1 (Photo 2-4-98) and on base lines B and 1 intersection, Level 1 (Photo 2-4-99). With some of the beams, lower parts of the rebars for the bottom reinforcement had fallen off due to explosion, and the rebars suffered a loss of cross-sectional area due to corrosion, as seen in Beam 2G1Aa, Level 2 (Photo 2-4-100) and Beam 2G3A, Level 2 (Photo 2-4-101). These damages parts require urgent repair, as corrosion and loss of rebars significantly affect the durability of the building. Rust fluid was also observed in some parts, which suggests that there may well be internal explosion. Causes for the explosion include salt damage as can be surmised based on the building's location, and for the inside of the warehouse, the affect by conditions during the period of time when it was used.

On Level 2, the roof had originally been made of wood at the time of its construction, while the brick walls were finished with mortar or tiles inside. The wooden roof had collapsed, however, and the inside of Level 2 was at present mostly exposed outdoors. The walls all had numerous cracks, many of which were large, measuring 2.0 mm or larger. As with the other parts of the building, the cracks were likely to have been caused by mortar shrinkage due to changes in temperature. One possible reason for the cracks being wider in these parts is because they were more exposed to direct sunlight, which would cause greater temperature swings, resulting in greater degrees of expansion and shrinkage. Another possible is that the base of the walls was bricks.

The eaves of Level 2 were in general rife with greater degrees of concrete cracks, explosion, falling-off, and corrosion. As seen in the eaves on base lines C and 2 intersection (Photo 2-4-102), the north-facing sides of the columns generally suffered a loss of cross-sectional area due to explosion, and in some parts, corrosion and a loss of cross-sectional area of reinforcement were observed and the column had entirely disappeared. The eaves themselves had also suffered explosion in some parts of the back side as seen in (Photo 2-4-103). The causes, again, are presumed to have been mortar shrinkage due to temperature changes, insufficient size of cover, and salt damage.

Given the loss of the main reinforcements of the columns which provide resistance to the horizontal load, and a large loss of cross-sectional area of the columns, the structure was at an extremely high risk of collapsing.

Photo 2-4-98. explosion and corrosion;
on base lines A and 2 intersection, Level 1

Photo 2-4-99. explosion and corrosion;
on base lines B and 1 intersection, Level 1

Photo 2-4-100. explosion and corrosion;
Beam 2G1Aa, Level 2

Photo 2-4-101. Cracks and falling-off;
Beam 2G3A, Level 2

Photo 2-4-102. explosion; Eaves on base lines C and 2 intersection

Photo 2-4-103. explosion; back side of eaves

①-2 Analysis of test results

The purpose of this survey was to examine the concrete structure for damage. To this end, compressive strength test and static elastic modulus test were performed on concrete, as well as testing for salt damage and neutralization, which can be causes of deterioration, while reinforcement in concrete were tested for the degree of corrosion. In addition, measurement of microtremor, wind speed and direction, temperature, and humidity, and a survey of the foundations were conducted.

In the tests on concrete, the compressive strength was lower than the original design specification of 13.5 N/mm^2 for the beams. The static elastic modulus, meanwhile, was lower than the original design specifications for the columns, beams, and slabs. The decline in concrete strength may have been due to the blend of materials made at the time of construction, or deterioration with age. The chloride ion content, meanwhile, was found to be higher at shallow levels. The chloride ion level exceeded the chloride threshold for corrosion where the rebars were, and rebar corrosion was also observed. Neutralization depth measurement, on the other hand, found little neutralization.

As for the concrete frames and rebars of the building, the progress of partial concrete deterioration and rebar corrosion is implied, even in areas where no explosion had occurred.

In the microtremor measurement, the natural frequency of the remains was 20 - 30 Hz, compared to 1 - 3 Hz for the typical predominant frequency of earthquakes. It may there be surmised that the building should not experience major resonance phenomena in the event of earthquake.

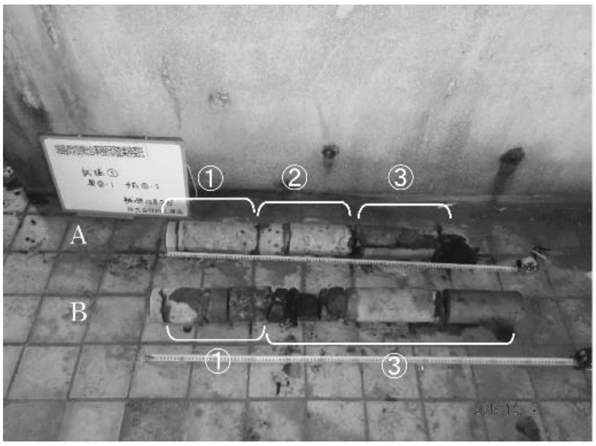
In the measurement of wind speed and direction, temperature, and humidity, the average wind speed was below 2.0 m/s in the morning and below 1.0 m/s in the afternoon, showing a trend toward calm winds. The wind direction remained mostly stable in the afternoon, largely southeasterly and southwesterly. Over the period of measurement of temperature and humidity, the average temperature was 20.3°C, and the average relative humidity was 62.9%RH; both remained stable.

①-3 Understanding the conditions of the foundations

In this survey, exploratory drilling was carried out to understand the conditions of the foundations and footing beams of the General Office building. At Exploratory Drilling Site ② (lower parts of Column on base lines A and 3 intersection), which was situated near the brick walls, gutters were found buried underground and the geometry of footings could not be determined. However, the cores drilled out vertically from the foot of the column (Area A) (Photo 2-4-104) and from where footing beams were estimated to be (Area B) (Photo 2-4-105) showed the existence of a layer of unreinforced concrete with a thickness of approximately 350 mm on the surface in both Areas A and B, as shown in (Photo 2-4-105). Underneath this layer were bricks and foundation stones which were presumably the foundations of the brick walls in Area B. In Area A, meanwhile, beneath the unreinforced concrete layer was a layer of concrete with a thickness of approximately 250 mm which was presumably the footings, under which there were foundation stones that were presumed to be the foundation of the brick walls. Similarly, a layer of unreinforced concrete with a thickness of approximately 350 mm and independent footings of unreinforced concrete with a thickness of approximately 400 mm were observed at Exploratory Drilling Site ①. Estimated cross-section views of the drilling sites are provided in (Figure 2-4-50).

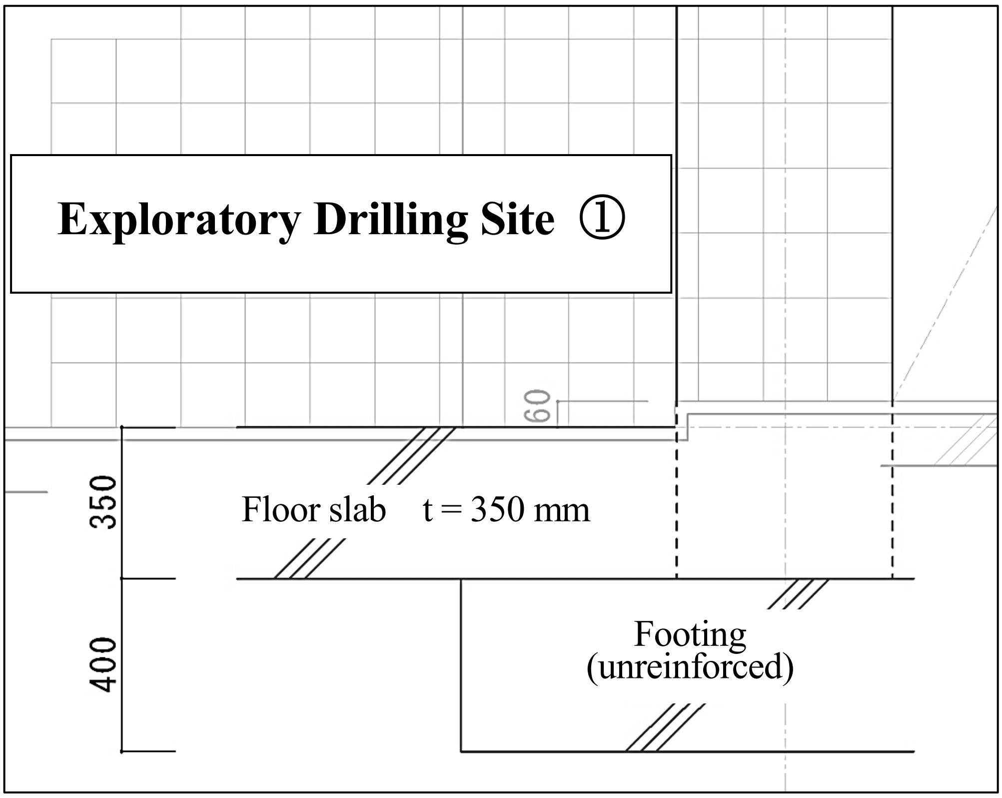
Based on these findings, it is surmised that the foundations of the building are independent, unreinforced footings. For the foundation of the columns where the building was connected to the brick walls, it is presumed that the footings were laid on top of the foundations of the brick walls, which had already existed and were left as is. There was no footing beams found, as a result of core boring and rebar exploration.

Photo 2-4-104. Core boring site; on base lines A and 3 intersection



① Floor slab 350 mm
② Footing
③ Foundations of brick walls

Photo 2-4-105. Cross-section view of drilled cores



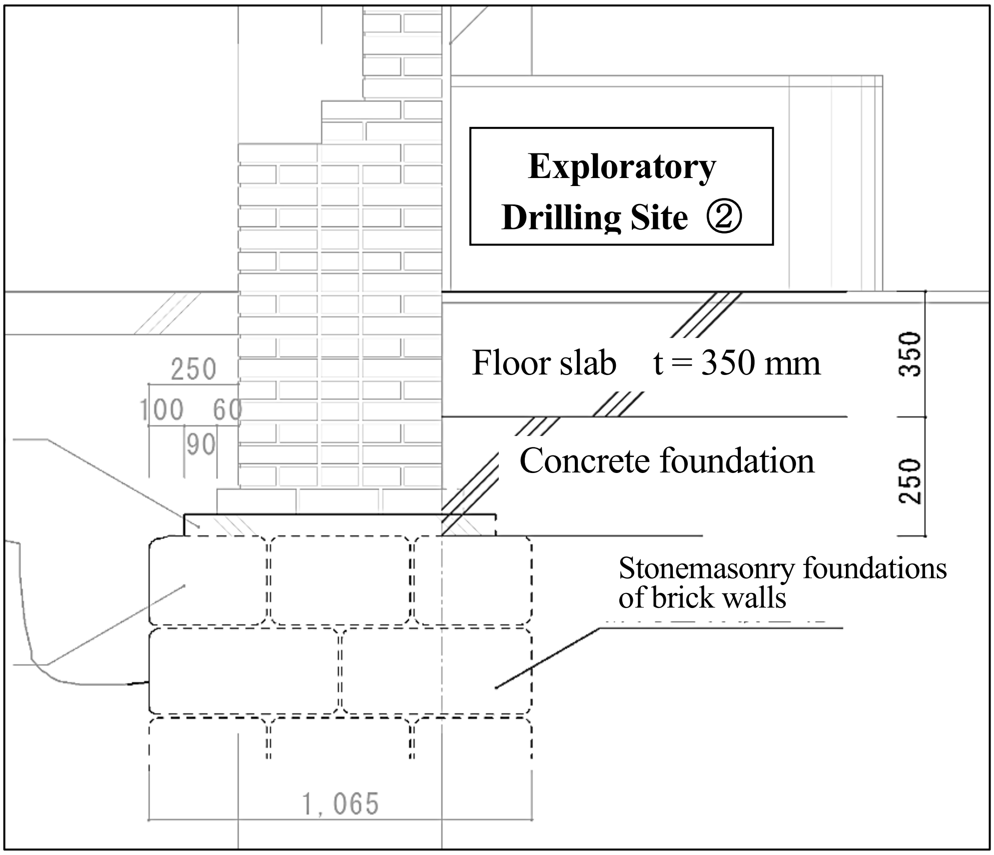


Figure 2-4-50. Estimated cross-section views of foundation exploratory drilling sites

①-4 Joint condition between the brick walls and the General Office

Based on the results of visual examination, it is surmised that the General Office building was mostly built after the brick-walled building had been built, where the structure of brick walls was left intact and the newer building was constructed so that it was completely jointed to the brick walls, without leaving clearance.

As for the foundations, the newer building was built with the foundations of the brick walls left intact, which resulted in the foundations of the brick walls bearing part of the load of the General Office building.

The beam in the General Office (Figure 2-4-51), meanwhile, was installed directly on the top of Arch No. 4 of the brick walls, as illustrated in (Figure 2-4-52) and (Photo 2-4-106). This leaves the upper part of the arch to bear the load.

Visual examination found that the slab supported by this beam showed signs of having been newly placed or re-cast. This indicates that the beam which was laid on top of the arch, too, would have been added at the same timing.

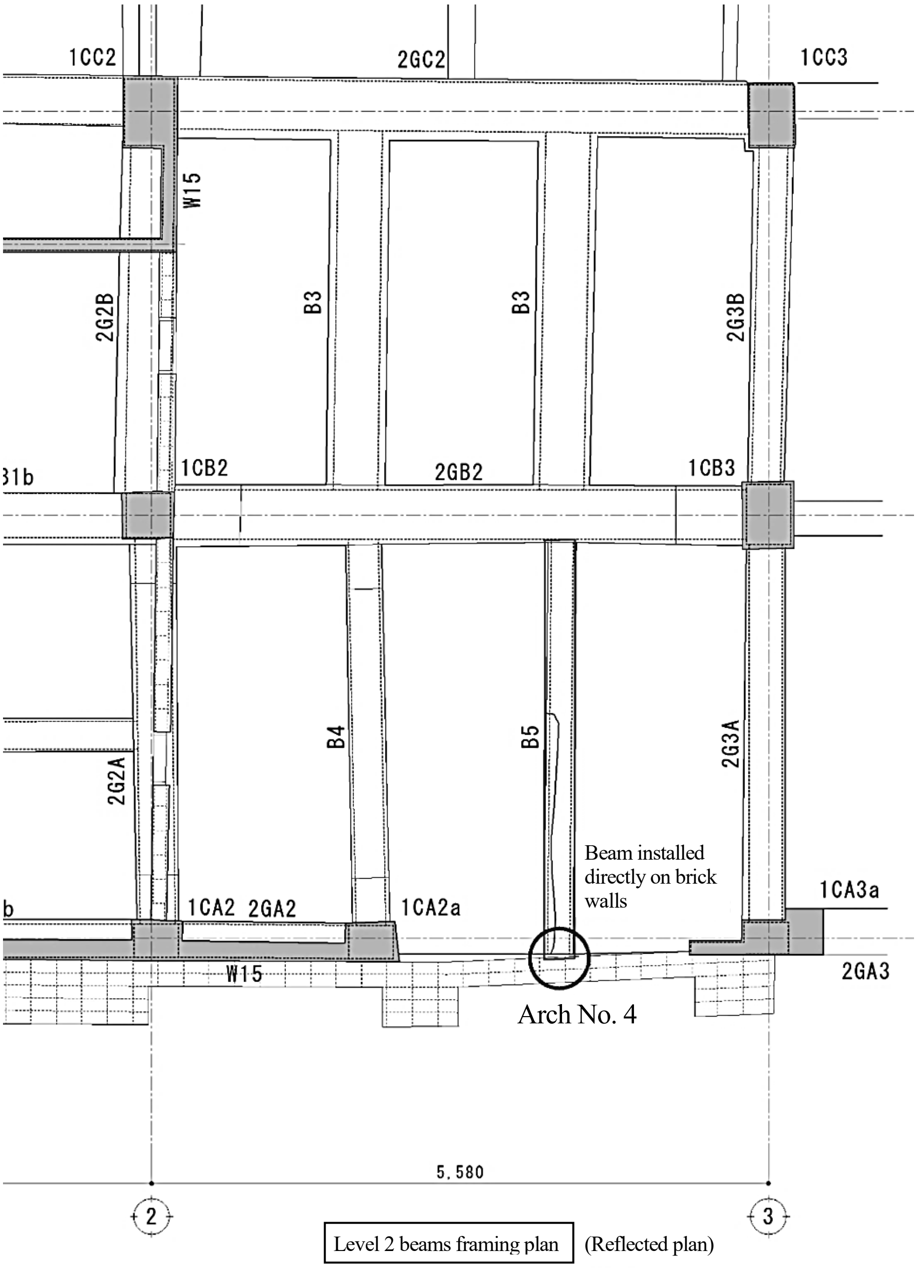


Figure 2-4-51. Level 2 beams framing plan

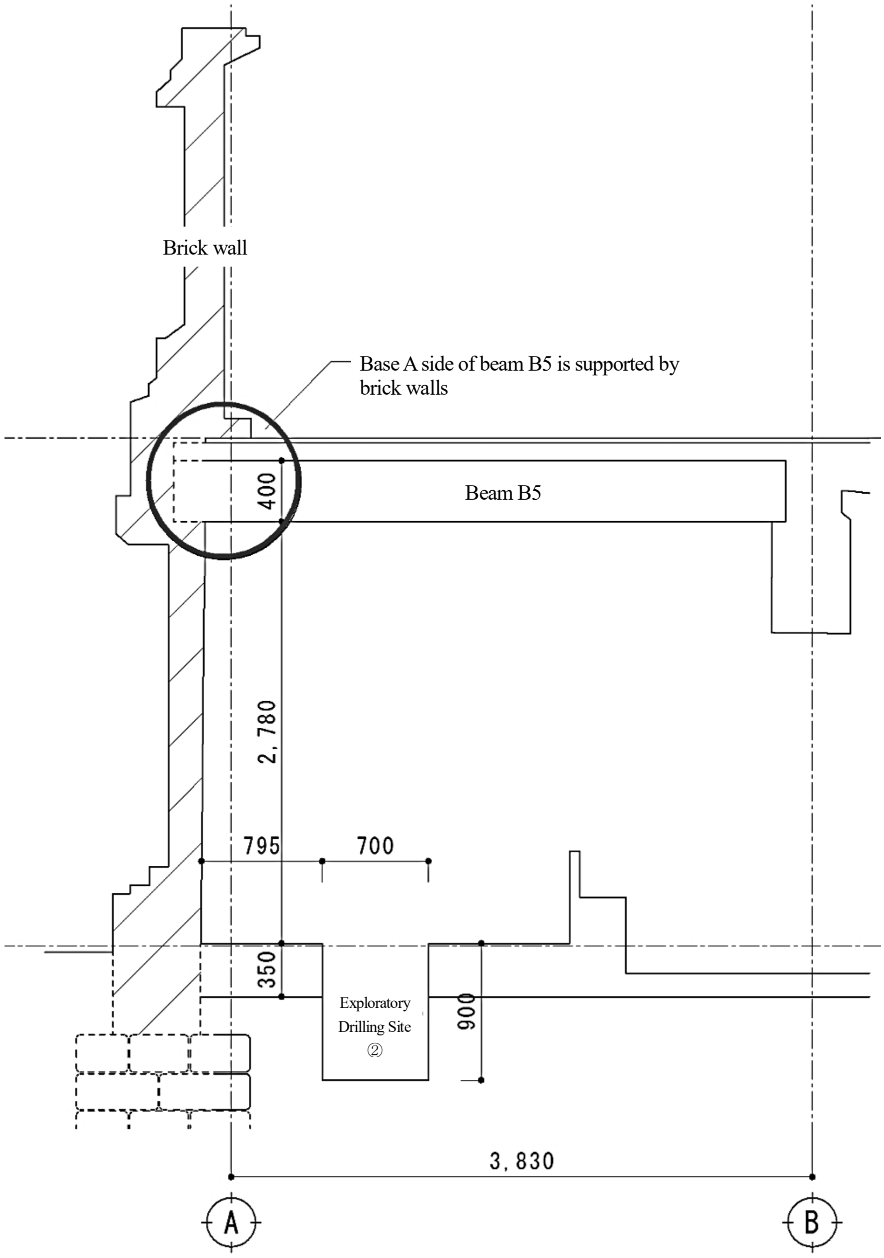


Figure 2-4-52. Relationship between beam and arch

Photo 2-4-106. Joint condition between Beam B5 and brick walls

①-5 Current conditions of damage and issues to be addressed

1. Considering that some parts of the concrete had lower strength than in other parts and that the chloride ion levels in the areas where the rebars were exceeded the chloride threshold for corrosion, there are possibilities of partial deterioration of concrete, even in parts where no explosion, etc., had occurred. In addition, parts of the rebars inside corroded, while some of the columns and beams suffered a loss of cross-sectional area in concrete and rebars. As shown in the drawings of damage, concrete around the areas where explosion had occurred was floating and in danger of eventually falling off. It is necessary to take measures to prevent concrete from falling off.

2. The eaves of Level 2 were in general rife with greater degrees of concrete cracks, explosion, falling-off, and corrosion. The north-facing sides of the columns generally suffered a loss of cross-sectional area due to explosion, and in some parts, corrosion and a loss of cross-sectional area of reinforcement were observed and the column had entirely disappeared. The collapse of the eaves may put the frame of the General Office building at risk of damage, and it calls for repair or other measures to protect the lower parts of the building frame against damage caused by possible collapse of the eaves.

3. The foundations of the columns and wall of the General Office building on the side that is connected to the brick walls were laid on top of the stonemasonry foundations of the brick walls. This means that the foundations of the brick walls have a high possibility to be under excessive strain. The foundations on base lines C and 3 intersection, in particular, is under load of the foundations of the columns in General Office as well as that of adjacent columns, which are considered to have been installed at the time of building expansion on the eastern side of base line 3. This was presumably the cause of the subsidence on the eastern side of the brick walls. The subsidence is considered to have been caused by structural disproportion rather than by weak ground, as there was no evidence of subsidence of the concrete floor slab on the side of the General Office building.

4. Arch No. 4 of the brick walls had large cracks in the lower parts, as shown in (Photo 2-4-107). Judging by the absence of subsidence or deformation with the crown of the arch itself and by the conditions of the joints on the surface of the walls with the cracks, the cause for these cracks is considered to have been the ground subsidence around the center, which caused the entire area of the walls surrounded by the white border in (Photo 2-4-107) to sink. The column on the right-hand side of the crown of the arch, meanwhile, appears to be leaning outward, as indicated by the blue arrow in Photo 2-4-107. This may have occurred as a result of the cracks in the wall; the cracks turned the part on the right-hand side of the arch into something similar to an independent masonry structure, which then were acted by stress of the arch. Furthermore, the part under the crown of the arch had some bricks missing as a result of deterioration of the joints with age, which may

eventually lead the arch itself to collapse. Because a collapse of the arch could in turn cause the brick walls to fall apart, this calls for measures to address this issue.

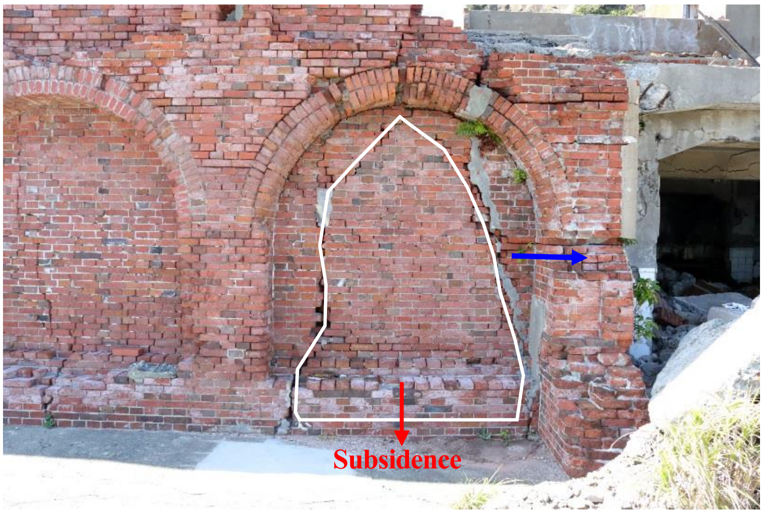


Photo 2-4-107. Damage to Arch No. 4 of brick walls

5. Beam B5, which was laid over base lines A and B and between base lines 2 and 3, was installed directly on the top of Arch No. 4 of the brick walls. This means that the floor load of the General Office building was put on the upper part of the arch, as shown in (Figure 2-4-53). Considering one side of Arch No. 4 was leaning outward as mentioned in 4. above, this load put to the arch may deform the arch even further, potentially leading it to collapse. Measures need to be taken to prevent such an eventuality, such as by removing the load on the beam.

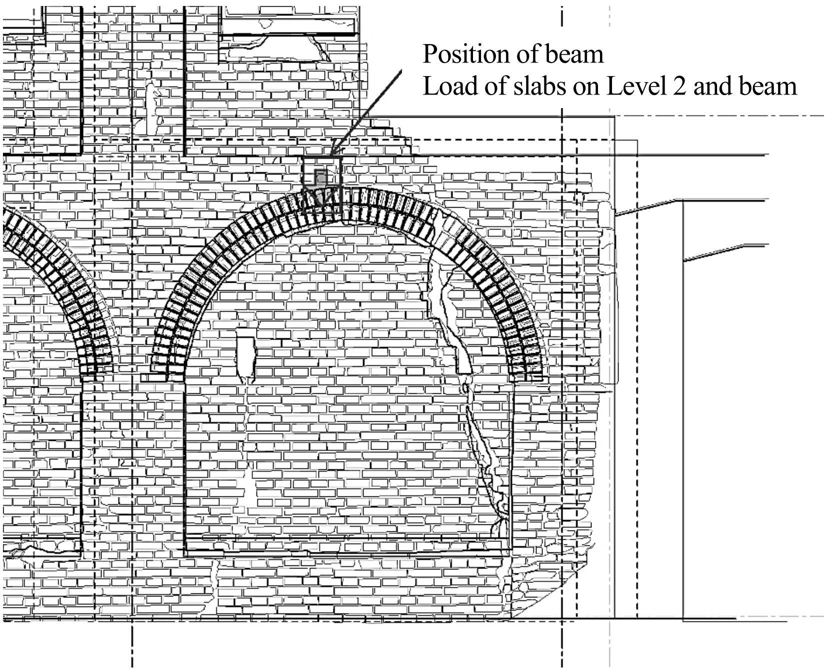


Figure 2-4-53. Drawing of damaged parts of brick walls

6. There was no underground beam laid in the General Office building. The rotational stiffness of the column bases on Level 1 was therefore low, and while they could resist horizontal force in the north-south direction as a portal moment frame, their horizontal load bearing capacity is considered small, given the loss of cross-sectional area in concrete and the loss of rebars in two of the girders that run in the north-south direction.

9) Rainwater infiltration at the top of the brick walls and measures

Visual examination was conducted on possible routes of rainwater infiltration into the brick walls, including the top of the brick walls, rain gutters, and the counters under the arch.

The top of the walls on Level 2 had a number of gaps created as a result of deterioration of joints, as shown in (Photo 2-4-108). The same could be said with the counters under the arch. On the western side, the joints became completely separated, with a few bricks fallen off at the end as shown in (Photo 2-4-109). On the eastern side, meanwhile, there was substantial damage to the bricks in the upper parts of Arch No. 4 with large cracks visible, as shown in (Photo 2-4-110); it is highly likely that rainwater went in through them. The mortar rain gutters, on the other hand, had no severe deterioration, and did not require repair.

Although it could not be determined in this examination whether or not rainwater was penetrating inside through the walls, considering that joints of the top of the walls were significantly deteriorated, rainwater infiltration was entirely possible. It will be necessary to repair the damaged joints and the large cracks, and make the wall waterproof.


Photo 2-4-108. Top of brick walls


Photo 2-4-109. Western side of brick walls


Photo 2-4-110. Eastern side of brick walls

Appendix 3