Tohoku ILC Civil Engineering Plan

October 2020

Tohoku ILC Project Development Center

In cooperation with

High Energy Accelerator Research Organization

Introduction

Since about 2014, research and examinations have been conducted by the relevant organizations around the proposed site for the civil engineering plan of the International Linear Collider (ILC) that is to be constructed in the Kitakami site in the Tohoku Region. In cooperation with the High Energy Accelerator Research Organization, the project now contains the full results from the consideration of the Global Design Effort for the ILC.

It is important for the technical feasibility of a project for an ILC facility to be evaluated technically in terms of rock mechanics and rock engineering because the facility requires large underground works. Accordingly, an ILC civil engineering plan in the Kitakami site in the Tohoku Region (hereafter referred to the “Tohoku ILC Civil Engineering Plan”) was examined for its technical feasibility by the Evaluation Subcommittee for ILC Civil Engineering Facility in Tohoku, established under the Committee for Rock Mechanics, the Japan Society of Civil Engineers (a public interest incorporated association), in the 2019 fiscal year. This subcommittee then evaluated the project with the comment of “We assessed the technical feasibility of the ‘Tohoku ILC Civil Engineering Plan’ as guaranteed, and concluded that the details of the project are appropriate.”

The Tohoku ILC Project Development Center opens the details of the construction technology of the civil engineering plan to the public and contributes to various scientific and technical examinations of it in many fields, considering these activities to be both very beneficial and essential for stepping toward the realization of the ILC. The development center has thus reorganized the technical details of the ILC civil engineering plan in the Kitakami site and has decided to introduce them here.

We expect that the ILC civil engineering plan will be widely understood through this document and the consideration of the technical details of it will be developed by many specialists in various fields.

Atsuto Suzuki,
Director of the Tohoku ILC Project Development Center

Contribution to disclosing the Tohoku ILC Civil Engineering Plan

The Tohoku ILC Civil Engineering Plan has been established by the Tohoku ILC Preparation Office in cooperation with the High Energy Accelerator Research Organization (KEK), and has been evaluated for its technical feasibility in terms of rock mechanics and rock engineering by the Evaluation Subcommittee for ILC Civil Engineering Facility in Tohoku in the Committee for Rock Mechanics, the Japan Society of Civil Engineers (a public interest incorporated association). We describe the circumstances of these activities and the result of the evaluation by the Japan Society of Civil Engineering below.

Committee for Rock Mechanics, the Japan Society of Civil Engineering
Evaluation Subcommittee for ILC Civil Engineering Facility in Tohoku

I. Circumstances

1. Tohoku ILC Civil Engineering Plan

The ILC project is based on the Technical Design Report (2013), and has been examined for cost reduction and other purposes in Japan by international cooperative works with researchers in many countries, with the High Energy Accelerator Research Organization (KEK) as the central figure. The specifications of the ILC project have been almost established up to now.

On the other hand, the Tohoku ILC Preparation Office has carried out point surveys, concept design, and other activities independently for the achievement of ILC location in the proposed area of the Tohoku Region, because the ILC project involves large underground work, and must consider some conditions intrinsic to its planned point, such as topography and geology.

Adding intrinsic conditions regarding the proposed point to internationally-examined specifications, the Tohoku ILC Preparation Office and KEK have established the Tohoku ILC Civil Engineering Plan described here.

2. Evaluation of the civil engineering plan

The feasibility of the civil engineering plan is required to be satisfactorily guaranteed in terms of rock mechanics and rock engineering, because the construction of the ILC facility involves large underground work. Accordingly, the evaluation of this civil engineering plan was entrusted to the Committee for Rock Mechanics, the Japan Society of Civil Engineering (a public interest incorporated association), which had also carried out technical examinations for ILC civil engineering plans as a third party in the past.

3. Evaluation Subcommittee for ILC Civil Engineering Facility in Tohoku in the Committee for Rock Mechanics, the Japan Society of Civil Engineering

3.1 Establishment of the subcommittee and member line-up

The Committee for Rock Mechanics decided to promote evaluation works by establishing the exclusive “Evaluation Subcommittee for ILC Civil Engineering Facility in Tohoku” under the committee. The subcommittee members were widely selected from members of the Japanese Society for Rock Mechanics (JSRM) with consideration for their special fields; we then decided to carry out work with an “all-Japanese” organization in the fields of rock mechanics and rock engineering.

The above-mentioned JSRM is the Japanese branch of the International Society for Rock Mechanics and Rock Engineering (ISRM), a global academy of rock mechanics and rock engineering, and is an academic research organization in cooperation with the Science Council of Japan. The JSRM mainly consists of four academies, which are the Japan Society of Civil Engineering, the Japanese Geotechnical Society, the Mining and Material Processing Institute of Japan, and the Japan Society of Materials Science; thus, most Japanese researchers and engineers involved in rock mechanics and rock engineering belong to the JSRM.

This subcommittee is headed by Chairman Ohnishi, who is an ex-vice president of the ISRM, along with the chief director (Ito) and two ex-chief directors (Obara and Kyoya) of the JSRM, and other experienced members as well as an all-Japanese member line-up consisting of those in the JSRM, becoming a reliable work team both domestically and internationally.

3.2 Subcommittee member list

Chairman	Yuzo Ohnishi	(Emeritus Professor in Kyoto University)
Chief Secretary	Takashi Kyoya	(Department of Civil and Environmental Engineering in the Graduate School of Engineering, Tohoku University)
Secretary	Tomoyuki Sanuki	(Department of Physics in the Graduate School of Science, Tohoku University)
	Nobuhiro Terunuma	(High Energy Accelerator Research Organization)
Member	Jun-ichi Kodama	(Division of Sustainable Resources Engineering in the Graduate School of Engineering, Hokkaido University)
	Takafumi Seiki	(Division of Social Design in the Graduate School of Regional Development and Creativity, Utsunomiya University)
	Masahiko Osada	(Programs of Environmental Science and Civil Engineering in the Graduate School of Science and Engineering, Saitama University)
	Tomochika Tokunaga	(Department of Environment Systems in the Graduate School of Frontier Sciences, the University of Tokyo)
	Kiyoshi Kishida	(Department of Urban Management in the Graduate School of Engineering, Kyoto University)
	Tomofumi Koyama	(Department of Safety Management in the Faculty of Social Safety Sciences, Kansai University)
	Shinichi Akutagawa	(Department of Civil Engineering in the Graduate School of Engineering, Kobe University)
	Hideaki Yasuhara	(Engineering for Production and Environment in the Graduate School of Science and Engineering, Ehime University)
	Yasuhiro Mitani	(Department of Civil and Structural Engineering in the Graduate School of Engineering, Kyushu University)
	Yuzo Obara	(Department of Civil and Environmental Engineering in the Faculty of Engineering, Kumamoto University)
	Takatoshi Ito	(Chief director of the Japanese Society for Rock Mechanics)
	Yoshinobu Nishimoto	(Chairman of the Committee for Rock Mechanics)
	Satoru Yamashita	(International Center for Elementary Particle Physics, the University of Tokyo)
	Shinichiro Michizono	(High Energy Accelerator Research Organization)

3.3 Activities of the subcommittee

Activities:

(1) Explanation of the outline of the ILC project and civil engineering plan

(2) Explanation of the Tohoku ILC Civil Engineering Plan

(3) Inspection of the proposed site and extraction of points to note

(4) Summarization of evaluations and points to note

Subcommittee meetings and on-site inspection:

- 1st meeting (July 30, 2019)
- 2nd meeting (August 30, 2019)
- 3rd meeting (November 11, 2019)
- On-site inspection (November 27, 2019)
- 4th meeting (December 19, 2019)
- 5th meeting (February 20, 2020)

The End.

Takashi Kyoya,
Chief Secretary of the Evaluation Subcommittee for ILC Civil Engineering Facility in Tohoku
in the Committee for Rock Mechanics, the Japan Society of Civil Engineering

Ⅱ. Result of the evaluation of the Tohoku ILC Civil Engineering Plan (extracted)[1)]

The Evaluation Subcommittee for ILC Civil Engineering Facility in Tohoku concluded that the "Tohoku ILC Civil Engineering Plan" is technically feasible and that the contents of the plan are appropriate.

The Evaluation Subcommittee for ILC Civil Engineering Facility in Tohoku investigated a wide range of topics covering the entire facility planning process as follows: 1) Description of the ILC Plan Summary and Civil Engineering Plan, 2) Description of Tohoku ILC Civil Engineering Plan, 3) Field survey of the candidate site and identification of points to be noted, and 4) Summary of evaluation and points to be noted.

The Subcommittee started the evaluation in mid-July 2019 and completed the work in February 2020. The features of this facility are that the main tunnel, through which the beams run, has a substantial total length of 20.5 km accompanied by five access tunnels (width 8 m, height 7.5 m, semicylindrical shape) and that there is a large cavern (width 25 m, length 108 m ~ 133 m, height 42 m) to house the detectors. Due to the various impacts arising from the large size of the facility and its underground location, it was necessary to take into account a wide range of perspectives such as rock engineering, geotechnical engineering, and hydrogeology, and to draw on the Japanese civil engineering technologies used in the past to construct tunnels and underground caverns as ordinary facilities, in order to assess the adequacy of the contents of the civil engineering plan.

Since the underground facilities will be constructed in underground rock with complex geological conditions, the following issues were discussed in this study: investigation of the unique characteristics of the proposed construction site; determination of the properties of the ground, rock, and water by surface and underground exploration; confirmation of the mechanical stability of the underground facilities for the construction safety; checking the impact of the design on the safe and rapid constructability of the facilities; and planning with consideration of economic efficiency.

The Subcommittee, consisting of leading experts in the relevant fields, conducted a careful and thorough evaluation based on the various exchanges of opinions and concluded that the "Tohoku ILC Civil Engineering Plan" is technically feasible from an expert's point of view and that the contents of the plan are appropriate.

February 20, 2020

Japan Society of Civil Engineers, Committee on Rock Mechanics

Evaluation Subcommittee for ILC Civil Engineering Facility in Tohoku

Chair Yuzo Ohnishi

[1)] The result of the evaluation can be viewed on the website of the Evaluation Subcommittee for ILC Civil Engineering Facility in Tohoku in the Committee for Rock Mechanics, the Japan Society of Civil Engineering (http://www.rock-jsce.org/index.php?ILC_subcommittee_2th).

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Part I

International Linear Collider (ILC)

1 Outline of the ILC Facility

1.1 The ILC accelerator

The configuration of the ILC (International Linear Collider) accelerator is shown in Figure 1.1[1, 2]. First, an electron source (gun) and a positron source located near the center of ILC generate a large number of electrons and positrons for use in the research of elementary particles. Several tens of billions of electrons and positrons form bunches and are injected into the damping rings. The bunch becomes a high-quality beam while it is circulating in the damping ring. The electron and positron beams from the damping rings are transported to the ends of the MLs (Main Linear Accelerators; Main Linacs) through the RTMLs (Rings To Main Linacs), and immediately accelerated by the MLs and transported to the collision point by the BDS (Beam Delivery System) which finally focus a bunch to an ultrasmall size of several nanometers for e+ e- collision. Elementary particle reactions that happen during the collision are completely recorded by the detectors installed around the collision point; then, the study of elementary particle physics is carried out. After passing through the collision point, electron and positron beams are sent to the beam dumps where they are absorbed.

A linear collider such as the ILC can be extended to perform experiments at further higher energies. This is a unique feature of a linear collider because a circular collider cannot change the circumference. The present ILC project is optimized for the research of the "Higgs boson" which is the most important and urgent research topic at present, with an energy of 250 GeV and a length of 20.5 km. The design allows for extension and upgrade in case the importance of high energy experiments is confirmed by the progress of ILC research.

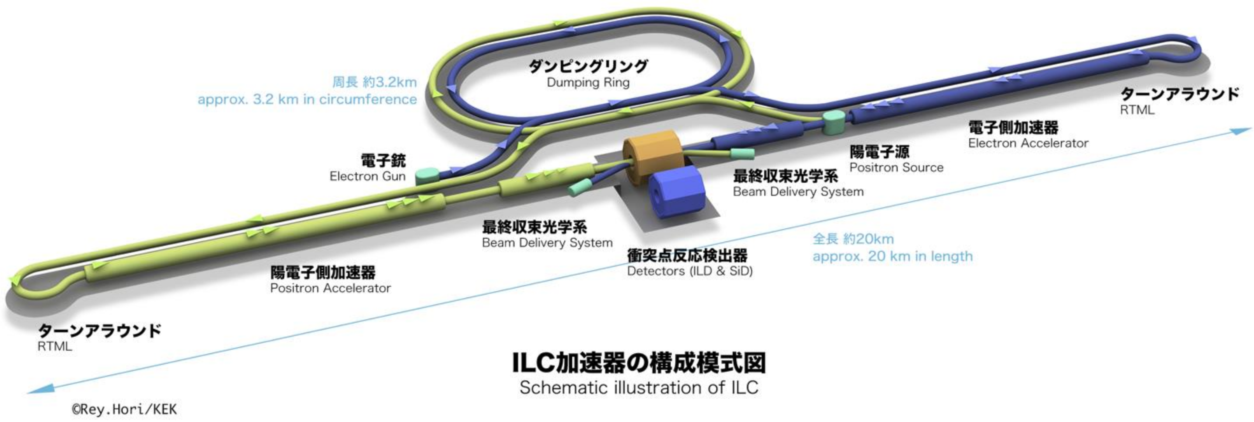


Figure 1.1. Configuration of the ILC accelerator.

1.2 ILC underground facility

Figure 1.2 shows an overview of the underground facility necessary for installing the ILC accelerator.

To install the ML and BDS equipment, the underground tunnels of the electron side and positron side are placed opposite each other. These tunnels are angled at 14 mrad in the horizontal plane to prevent beams from entering their opposing accelerator after passing through the collision point. Looped tunnels are required at both ends of ILC to accommodate the turnaround. In the central region, a tunnel with a race-track shape for damping rings and a detector hall for two detectors will be constructed.

Five access tunnels are used (one in the center and two each on the electron and positron sides) to connect the underground tunnel to the surface. The location of access tunnels to the underground tunnels is determined by the design of the accelerator. The detector hall will be connected to the surface by two shafts.

The length of the electron side is 10.9 km and that of the positron side is 9.6 km; accordingly, the overall length is 20.5 km. Since the electron side contains an accelerator system for the positron generation, the length of it is longer than that of the positron side.

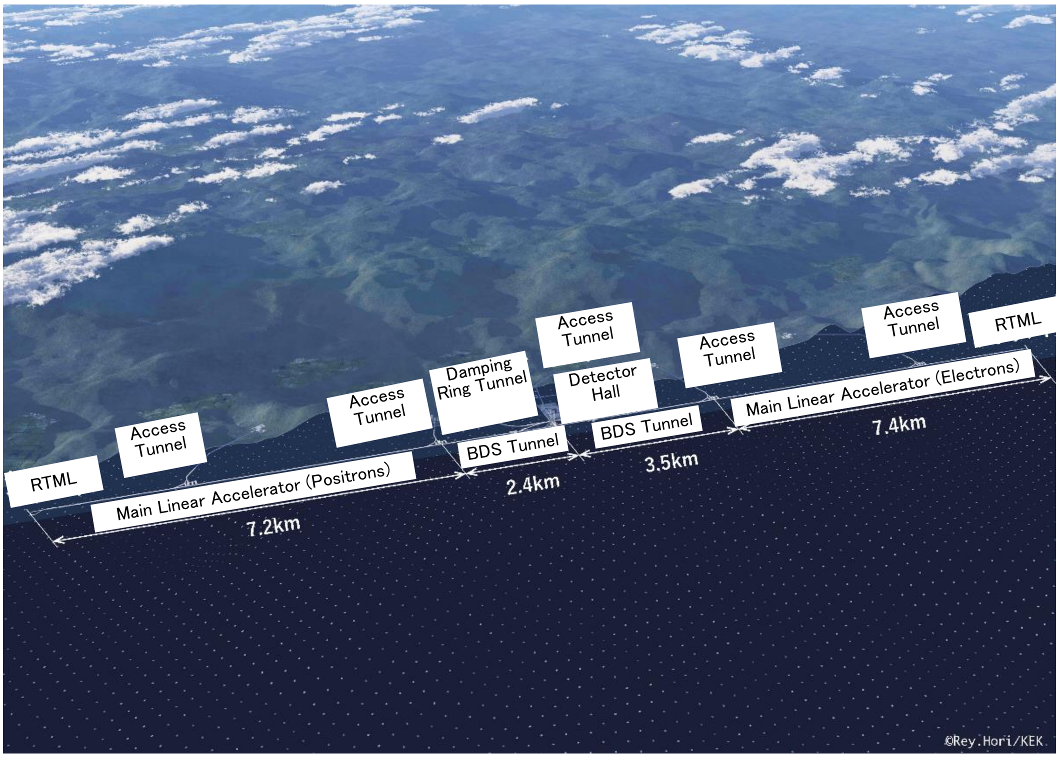


Figure 1.2. Configuration of the ILC underground facility.

2 Design Requirements

2.1 Access tunnels and access halls

The surface facility and underground accelerator tunnel are connected by an Access Tunnel (AT), which is an inclined tunnel for entry. There are two access tunnels connected to the electron side (AT-8, AT-10), two to the positron side (AT+8 and AT+10), and one to the damping ring and detector hall (AT-DR/DH)[2]. At the connection between the access tunnel and the accelerator tunnel, an Access Halls (AH) will be constructed to install the various kinds of equipment necessary for ILC operation (see Figure 2.1).

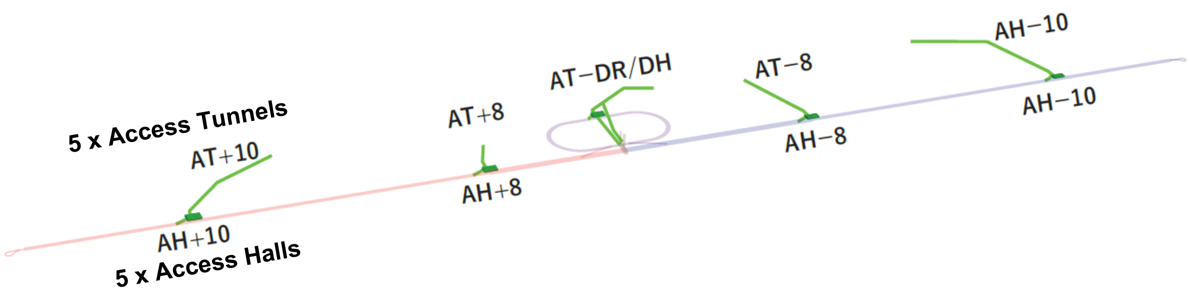


Figure 2.1. Access tunnels.

The access tunnels are used for carrying in large devices such as cold boxes and laying various kinds of cables, pipes, and ducts necessary for ILC operation (see Figure 2.2). In consideration of these uses, the cross section of an access tunnel is determined to be a semicylindrical (horseshoe) shape with a width of 8 m and a height of 7.5 m. The maximum slope of the access tunnel is 10% (9% on average) in accordance with the maximum longitudinal slope of 12% (a special value for the Types 1, 2, and 3 regular roads) stated in the Road Structure Ordinance.

The pithead of the access tunnel is located at the Access Stations (AS) which requires yards with an area of 18,200 m^2 for installing various kinds of equipment such as electrical and mechanical facilities. These AS yards are also used as temporary yards during construction.

The access hall consists of four domes (called an M dome, E dome, He dome, and S dome respectively) that contain the machinery, electric power equipment, plant for liquid helium, and various service equipment (see Figures 2.3 and 2.4).

The distance between each access halls and the collision point is determined by the design of the ILC accelerator.

[2] The numbers (-8, -10, +8, and +10) added to the access tunnels are based on the "Technical Design Report [1]."

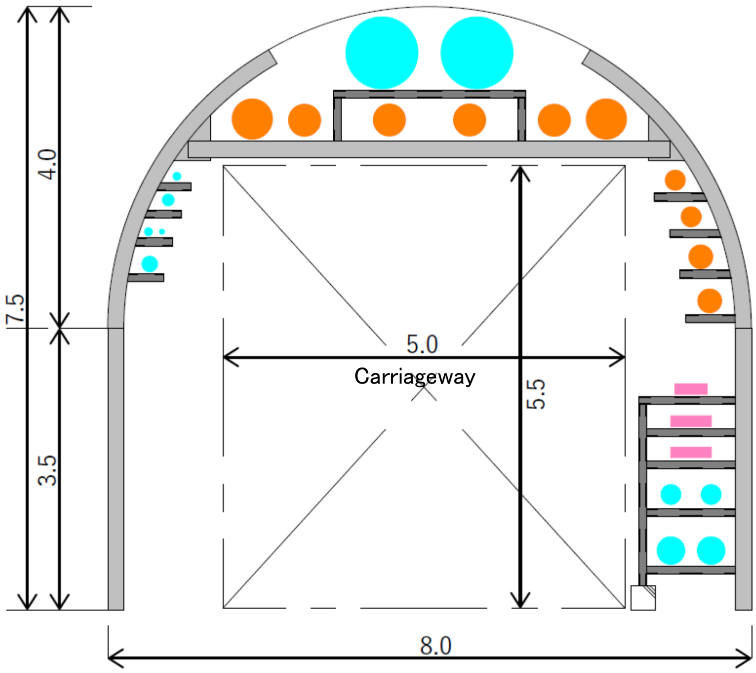


Figure 2.2. A plan of piping and cabling in the access tunnel (under consideration).

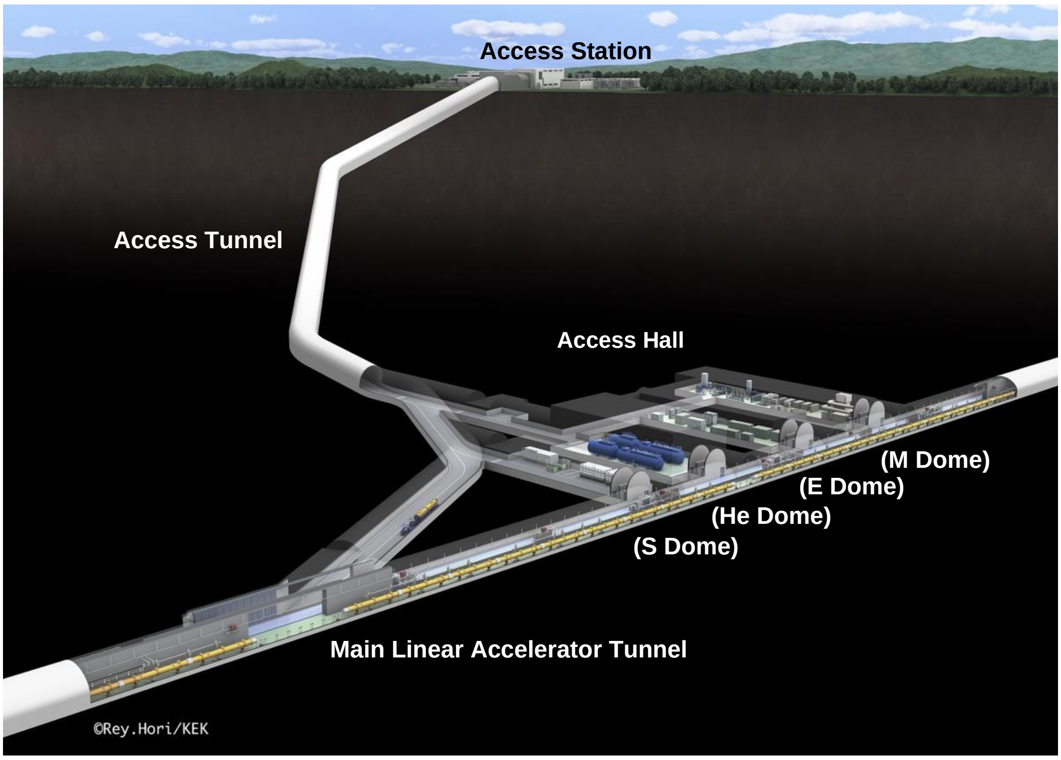


Figure 2.3. An illustration of a connection from an access station to a main linear accelerator tunnel.

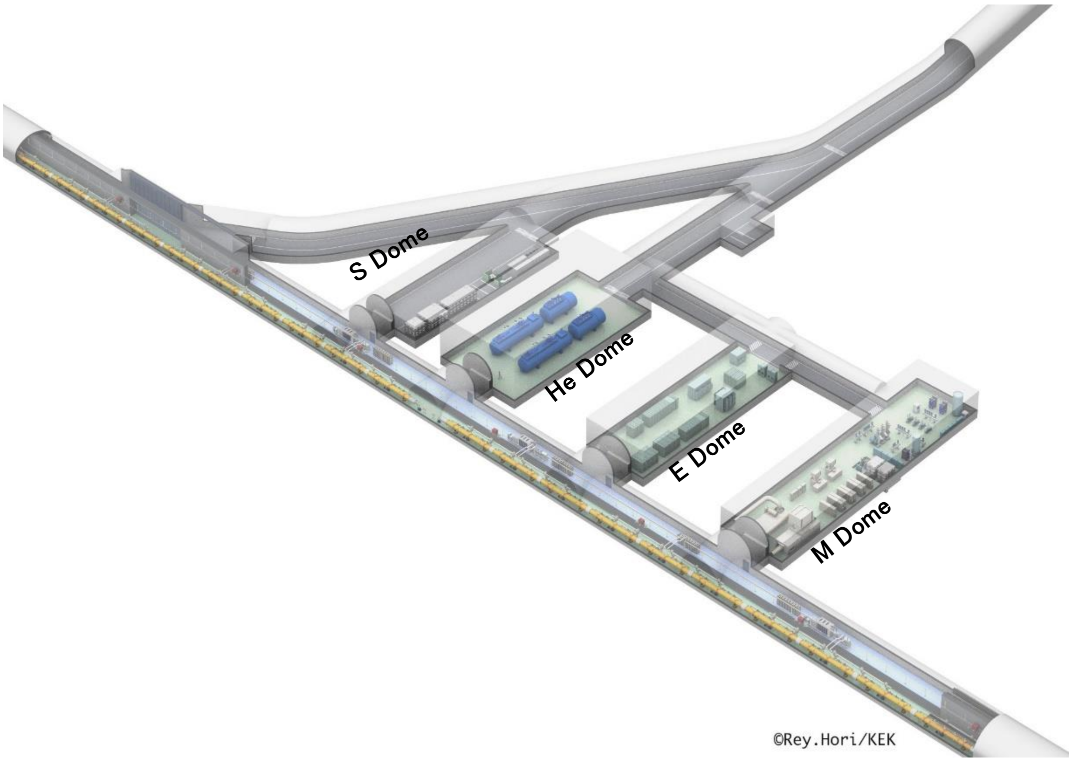


Figure 2.4. An illustration of the access hall.

2.2 Accelerator tunnel

The accelerator tunnel consists of the damping ring tunnel, main linear accelerator tunnels, BDS tunnels, and other tunnels connecting them.

2.2.1 Damping ring tunnel

A tunnel containing damping rings is placed in the central region and has a race-track shape combining straight and curved sections (see Figure 2.5). The circumference of this tunnel is 3.2 km.

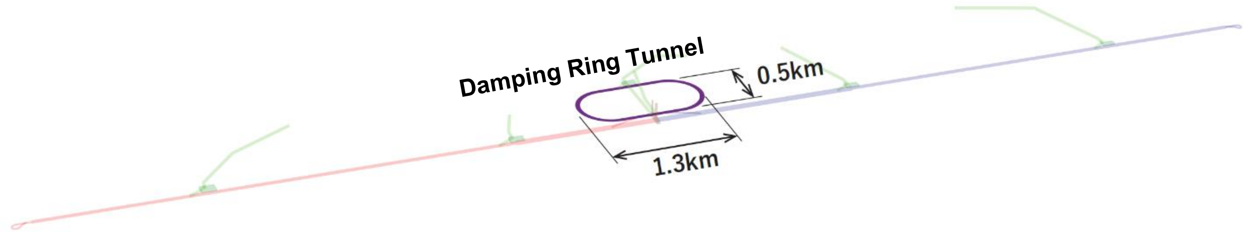


Figure 2.5. Damping ring tunnel.

In the damping ring tunnel, an electron damping ring and a positron damping ring are installed in two layers (see Figure 2.6). The inner space should be large enough to install the third ring for positron to cure the positron beam performance in future upgrade.

The curved sections of the damping ring tunnel contain longer bending magnets that require an inner space with a width of 5.5 m and a height of 4.7 m or more. On the other hand, the straight sections contain superconducting acceleration modules (cryomodules) and high-frequency generators, which require a larger inner space with a width of 11 m and a height of 5.5 m or more, and a concrete shield wall will be installed between them.

Figure 2.6. An illustration of the inside of a curved section of the damping ring tunnel.

2.2.2 Main linear accelerator tunnel

The main linear accelerator, which accelerates electron and positron beams to the required energy, will be installed in opposing straight tunnels (see Figure 2.7). In the current plan, which aims for 250 GeV collision energy, the lengths of the main linear accelerator tunnels are 7.4 km for the electron side and 7.2 km for the positron side.

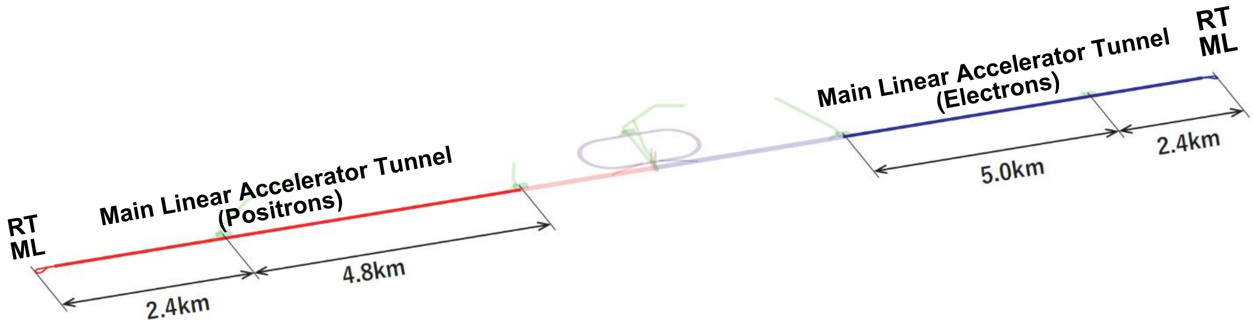


Figure 2.7. Main linear accelerator tunnels.

The main linear accelerator tunnels contain superconductive high-frequency accelerator modules (cryomodules) and high-frequency generators. The yellow cylindrical objects in Figure 2.8 are cryomodules. Concrete shield walls with a thickness of 1.5 m will be installed in the center of the tunnel for separating cryomodules and high-frequency generators. In addition to that, power lines, cooling water piping, and others necessary for the operation of high-frequency generators are installed. A large space also needs to be ensured for the installation and exchange of cryomodules and other devices. Based on the above conditions, the inner space of the main linear accelerator tunnel is determined to be a semicylindrical (horseshoe) shape with a width of 9.5 m and a height of 5.5 m.

In the turnaround section, bending magnets are installed to bend the beam path. A semicylindrical (horseshoe) shape tunnel with a width of 4.5 m and a height of 4.0 m is planned because a concrete shield wall is not necessary.

In addition, the main linear accelerator tunnel should be parallel to the geoid plane because a liquid helium is used in the cryomodules.

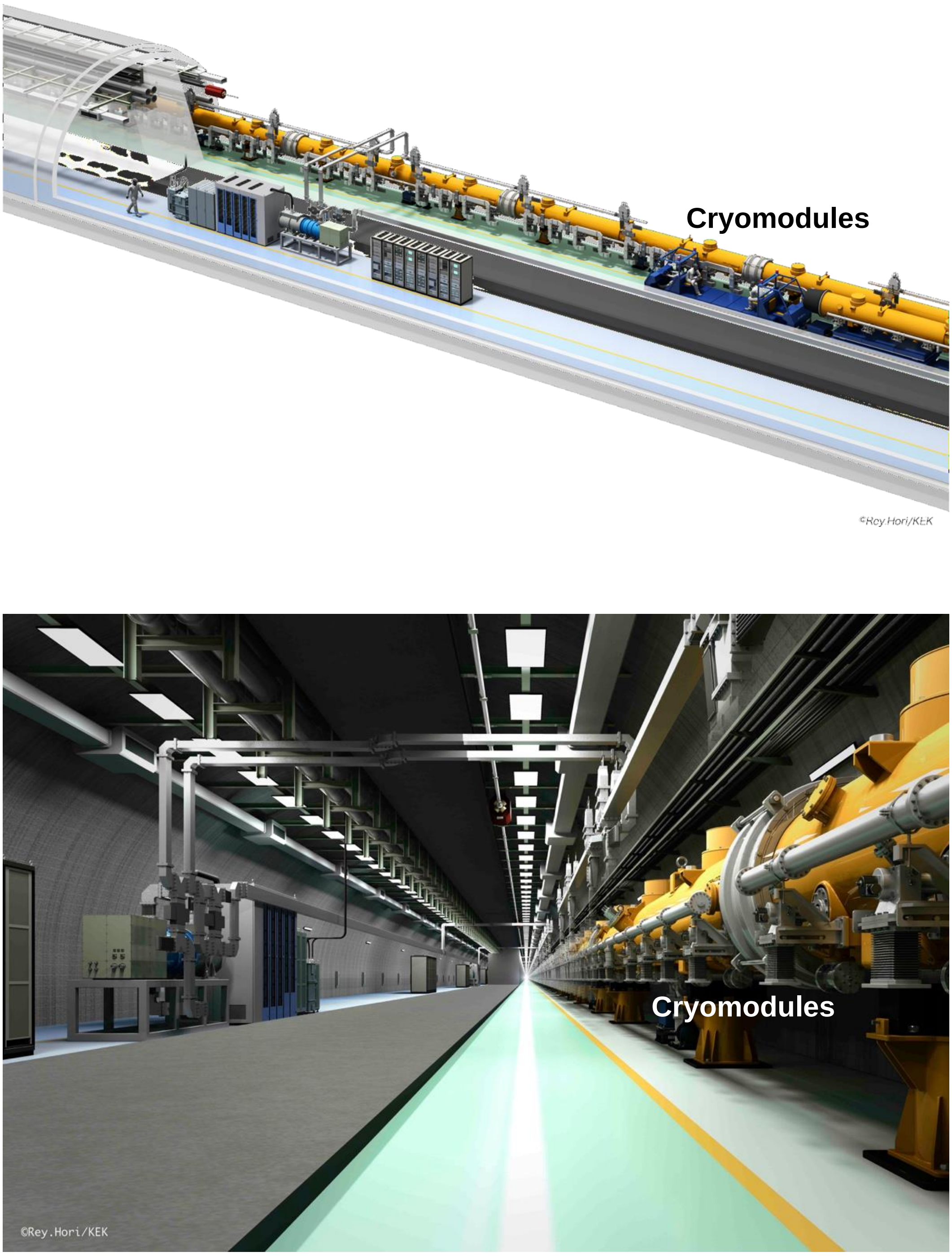


Figure 2.8. An illustration of the inside of a main linear accelerator tunnel (shield walls are not shown).

2.2.3 BDS tunnels

The Beam Delivery System (BDS) transports the electron and positron beams accelerated by the main linear accelerator to the collision point. The BDS section has two parallel tunnels[3)]: one is a beam tunnel that contains a group of magnets forming a BDS beamline, and the other is a service tunnel that contains power supplies and other equipment necessary for the beamline. The lengths of the BDS tunnel is 3.5 km for the electron side and 2.4 km for the positron side (see Figure 2.9).

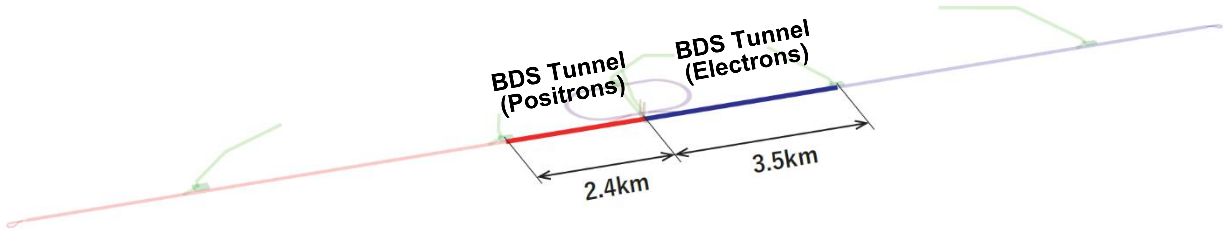


Figure 2.9. BDS tunnels.

The BDS beamline mainly consists of electromagnets. Figure 2.10 shows the ATF2 beamline constructed at KEK to demonstrate the BDS technology for ILC. The design of the BDS beamline for ILC is an origin of the ATF2 beamline though the sizes of them are different by beam energies. In addition, the BDS tunnel contains the electron/positron source as well as a BDS beamline. Since a cross section is required to be large due to inconstant distance between these beam lines, a semicylindrical (horseshoe) shape with a width of 8.0 m and a height of 5.0 m is planned. A cross section of the service tunnel is planned to be a semicylindrical (horseshoe) shape with a width of 4.5 m and a height of 4.0 m, which is large enough to contain power supply devices for electromagnets[4)].

To make small electron and positron beams which collide with sufficiently high repetition rate, it is important for a beam path not to be bent in a vertical direction (to make a beam path straight) in a BDS beamline. Thus, the BDS tunnels are constructed in laser straight unlike the main linear accelerator tunnels which constructed parallel to the geoid plane (see Figure 2.11).

3 These tunnels are integrated into one tunnel in the present design from the result of examinations by the Global Design Effort for the ILC.
4 A cross section of the integrated tunnel is a semicylindrical (horseshoe) shape with a width of 11 m and a height of 5.5 m.

Figure 2.10. ATF2 beamline to demonstrate the BDS technology.

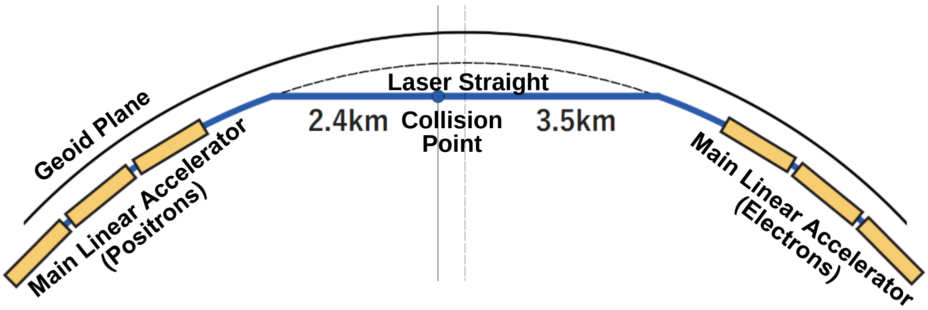


Figure 2.11. Relative position of the ILC accelerator to the geoid plane.

2.3 Detector hall and peripheral tunnels

A hall containing detectors is constructed at the point where the electron and positron beams collide. In this detector hall, two detectors will be installed on each movable base which brings the detector to the collision point alternately. Various kinds of machinery such as an air conditioner are also installed in the hall.

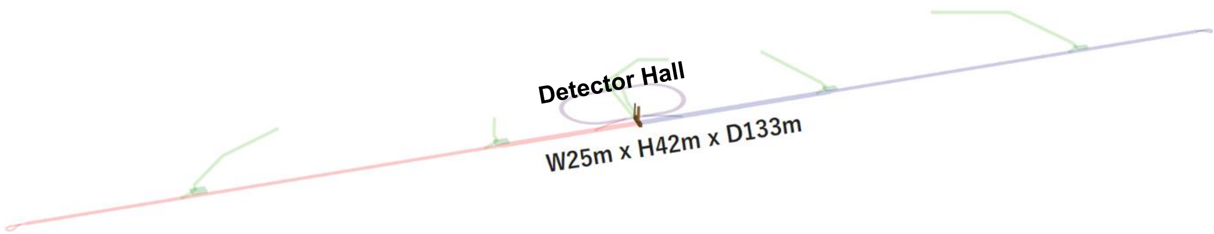


Figure 2.12. Detector hall.

The inner space necessary for detectors is determined to have a width of 25 m, a length of 108 m, and a height of 42 m. In addition to that, the space is planned to be extended by 25 m to contain machinery and other devices, and resulted an underground cavern with a length of 133 m.

The underground detector hall and surface will be connected two shafts, a shaft 18 m in diameter used for carrying the units of detectors, a shaft 10 m in diameter laying power lines and various kinds of pipes, and an access tunnel (see Figures 2.13 and 2.14). Peripheral tunnels of detector hall have sections with widths of 8 m and 4 m according to applications.

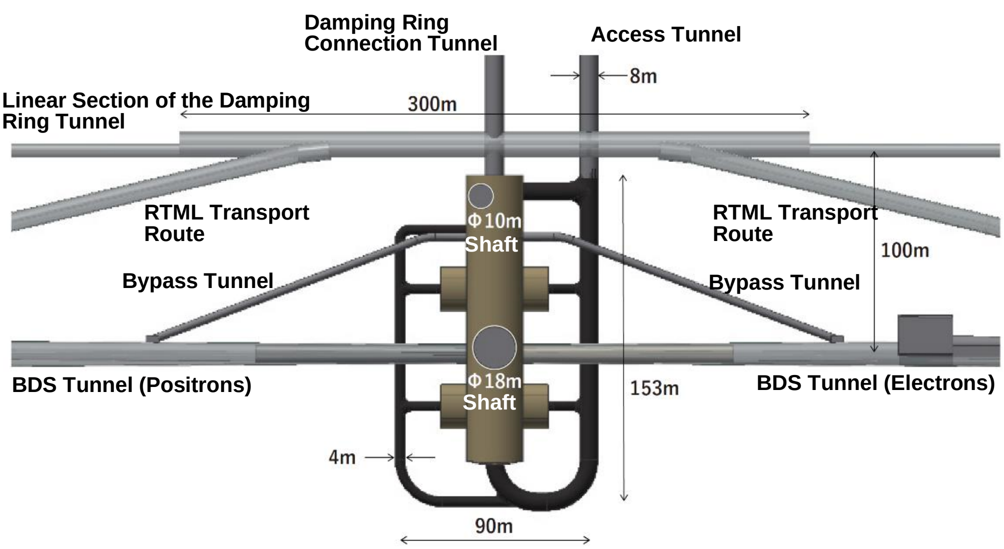


Figure 2.13. Plan view of the periphery of the detector hall.

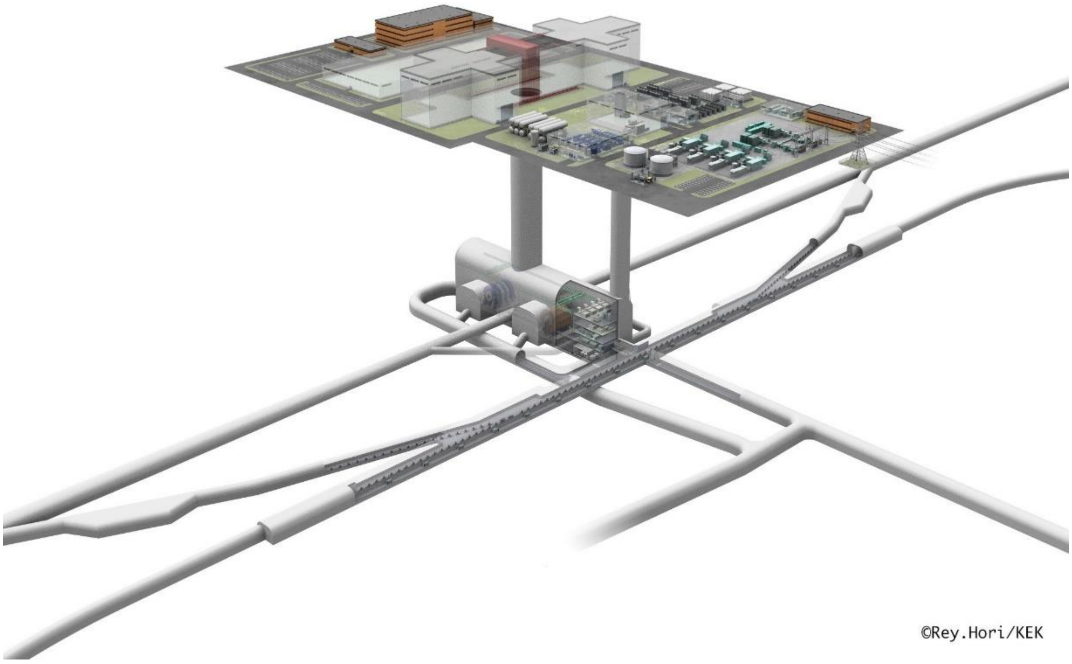


Figure 2.14. An illustration of the detector hall and peripheral tunnels.

Part II

Kitakami Site

3 Topography and Geology

3.1 Outline of topography

The Kitakami Mountains are a non-volcanic mountain region that extends 250 km north to south and 80 km east to west. Having ridges of constant height, the mountains are considered to be an uplifted peneplain with topographical features showing highland-like topography with gentle rises and falls. Though the Northeastern Japan Arc has some developed structural basins, the Kitakami Mountains do not seem to have them. In addition, the mountains do not have large rivers, and valleys along the rivers in the mountains are generally wide and gentle.

The planned area for the ILC is in the southern part of the Kitakami Mountains. Figure 3.1 (left) shows a topographic map of the Kitakami site [3]. Some topographical features of the site include belt-shaped topography extending from the Hitokabe region along the Toriumi River and drop-shaped topography extending from the Okita region to Fujisawa-cho. These types of topography correspond to distribution areas of granites belonging to the Hitokabe and Senmaya plutonic rock bodies, and form a characteristic topography that borders a granite rock body that is weathered easily due to hard hornfels formed around the granite rock body by contact metamorphism during intrusion. On the western side of the planned area for the ILC is the Kitakami River, on the west edge of the Kitakami Mountains, flowing down from north to south. Since many rivers in the planned area are tributaries of the Kitakami River, most of them flow down in a meandering way from east to west. These rivers mainly include, from the north, the Hitokabe River, Ide River, Satetsu River, and Senmaya River; on the other hand, the Kamiyama River and Tsuya River near the southern end run to the SSE and flow into the Pacific Ocean. Many mountains with an altitude of 700 to 900 m are formed along the planned area for the ILC. With an altitude of 782 m, Mt. Abara is the highest mountain among those formed by granite. Since other mountains seem to have a height of 400 to 500 m, they have a gentle highland-like topography as a whole. Around the circumference of the granite is a distribution of Mesozoic/Paleozoic strata affected by thermal metamorphism and many mountains with an altitude of 700 to 800 m. These mountains mainly include, from the north, Mt. Tenguiwa (an altitude of 775 m), Mt. Hourai (788 m), and Mt. Tokusenjou (712 m); in addition, Mt. Murone (895 m) is the highest mountain in the vicinity and is formed by the intrusion of dike swarms into Paleozoic strata and the Orikabe plutonic complex.

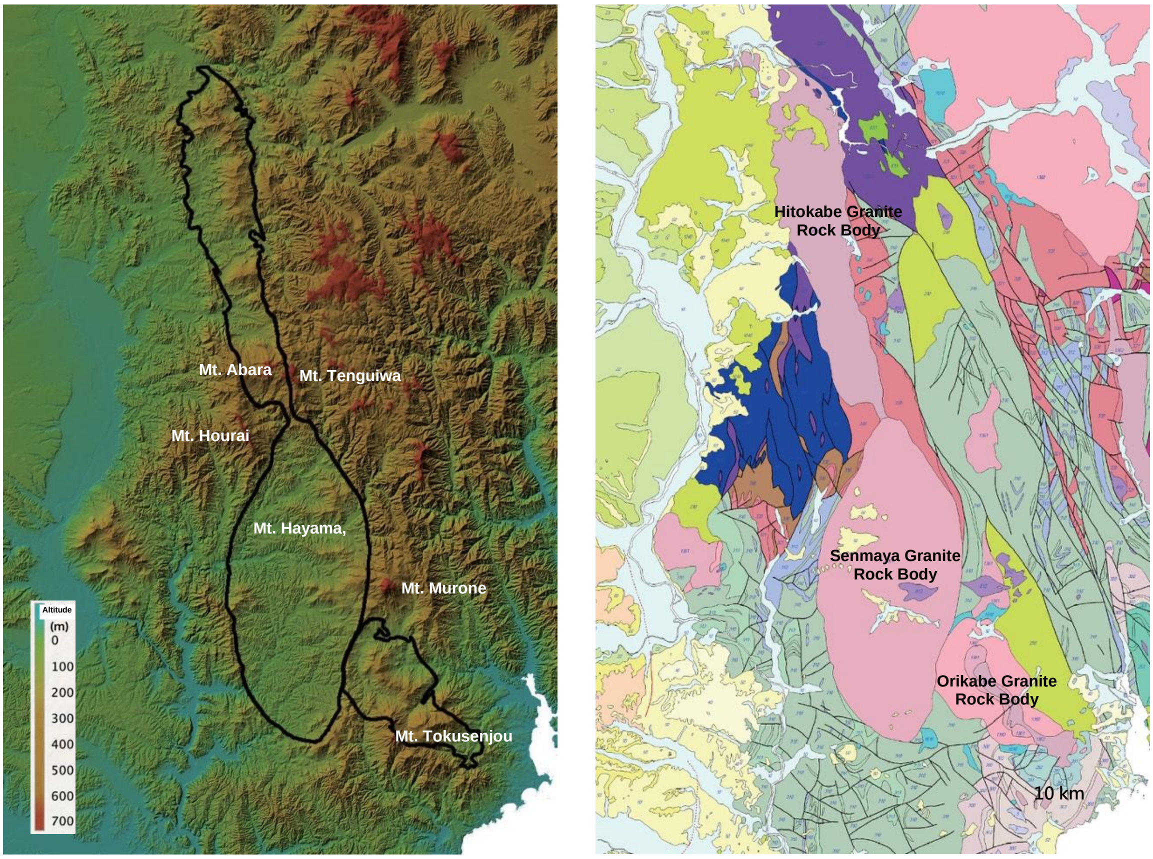


Figure 3.1. A topographic map (left) and a geologic map (right) of the Kitakami site.

3.2 Outline of the geology

The Kitakami Mountains are mainly formed by Paleozoic/Mesozoic sedimentary rocks and Cretaceous granite intruding into them. The mountains are geologically separated into the Northern and Southern Kitakami Mountains by a northwest-southeast border, the Hayachine Tectonic Belt extending from Morioka to Kamaishi. The planned area for the ILC is included in the Southern Kitakami Mountains. The distribution of geology in the Northern Kitakami Mountains is separated into the west Kuzumaki-Kamaishi and east Akka-Tanohata belts; while both belts mainly consist of Mesozoic/Paleozoic strata including allochthons such as cherts, the distribution of geology in the Southern Kitakami Mountains consists of neritic sedimentary rocks with pre-Silurian as a basement rock. In addition, many plutonic rock bodies mainly formed by granite intrude into these types of geology.

Figure 3.1 (right) shows a geologic map of the Kitakami site [4]. Extending up to 50 km, the ILC is planned to penetrate the Hitokabe plutonic rock body, Senmaya plutonic rock body, and Orikabe plutonic complex which extend into the southwest part of the Southern Kitakami Mountains, and to reach the Mesozoic Triassic Inai Group in an area south of the site. North of the site, older exposed geology surrounding granite can be found, and the vicinity of Miyamori and Motai has basic rocks and metamorphic rocks of the pre-Silurian period, which is a base rock of these regions. On the other hand, on their southern side is Carboniferous rock contacting the Hitokabe and Senmaya plutonic rock bodies. In addition, the Permian rock is distributed around the Carboniferous rock, Hitokabe plutonic rock body, and Orikabe plutonic complex while Triasic rocks such as the Inai Group are distributed to the south of it. The Carboniferous and Permian rock formations consist of lutite and sandstone inserted into limestone while the Triasic consists of mudstone, sandstone, and alternating strata. The Southern Kitakami Mountains have a fault structure extending from the Hizume region to the Kesennuma region in the NNW-SSE direction; this structure is called the Hizume-Kesennuma Fault. Fold structures in the same direction are widely developed in the Southern Kitakami Mountains, and were supposed to be formed by the Oshima Orogenic Movement activated mainly in the Cretaceous period. The Oshima Orogenic Movement is considered as a series of the following movements: the addition of the allochthonous strata of the Northern Kitakami Mountains occurred during the time from the middle Jurassic to the early Cretaceous periods due to compression in the east-west direction; and then, fold structures and faults in the same direction were formed due to volcanic activity and the intrusion of plutonic rocks. Alluvia and terrace deposits are distributed as younger sediments in valleys along rivers in the site; in addition to that, the unconformity of the Pliocene covers a part of the western Senmaya granite rock body.

4 Geological Surveys[5]

4.1 Description of geological surveys

The geological surveys were conducted almost along a straight line through the Hitokabe granite rock body, Senmaya granite rock body, and Orikabe granite rock body. Table 4.1 outlines the geological surveys conducted until now. In addition, Figure 4.1 shows the main survey points along with outlines of their results. As of the 2017 fiscal year, we have carried out seismic exploration with an overall length of 30 km, electromagnetic/electric prospecting with an overall length of 13 km, and boring surveys in six places (seven cores).

Table 4.1. Outline of the geological surveys

Fiscal year	Main survey item	Description of the survey	Ordered by
2009	Ground surface/geology investigation Seismic exploration Radiation survey	Investigation of the contact point ("narrow point") of the Hitokabe and Senmaya granite rock bodies	Iwate Pref.
2010	Ground surface/geology investigation Seismic exploration Electromagnetic prospecting Boring survey	Investigation of the properties and conditions of the Hitokabe and Senmaya granite rock bodies Investigation of the cross points of rivers	Tohoku Univ.
2011	Ground surface/geology investigation	Investigation of overviews of Mt. Hayama and the Inai Group	Tohoku Univ.
2012	Airborne laser survey	Creation of topographical maps (with a contour of 1m/5m) from the DEM (with a grid of 1m)	Tohoku Univ.
	Ground surface/geology investigation Seismic exploration Electromagnetic prospecting Boring survey	Investigation of the properties and conditions of geology around the proposed collision point Investigation of geologic boundaries Reading of topography (Inclined shafts are assumed for accessing the detector hall.)	Tohoku Univ.
	Ground surface/geology investigation	Investigation of lineaments and active faults	Iwate Pref.
2015	Ground surface/geology investigation Seismic exploration Electromagnetic prospecting Boring survey	Investigation of the properties and conditions of geology around the proposed collision point (Shafts are assumed for accessing the detector hall.)	Tohoku Univ.
2016	Seismic exploration	Investigation of the properties and conditions of the northern slope of Mt. Abara	Tohoku Univ.
2017	Seismic exploration	Whole line of 30 km	Tohoku Univ.

[5] The outline of the geological surveys conducted up to the 2017 fiscal year is described in material (Material 1-3 for the 6th Subcommittee Meeting for Technical Examinations held on October 2, 2018) reported to the "Committee of Examinations for the Reworked Plan of the ILC Project, the Science Council of Japan" (http://www.scj.go.jp/ja/member/iinkai/ILC/ILC24.html). In this document, the details of the civil engineering plan are simply summarized.

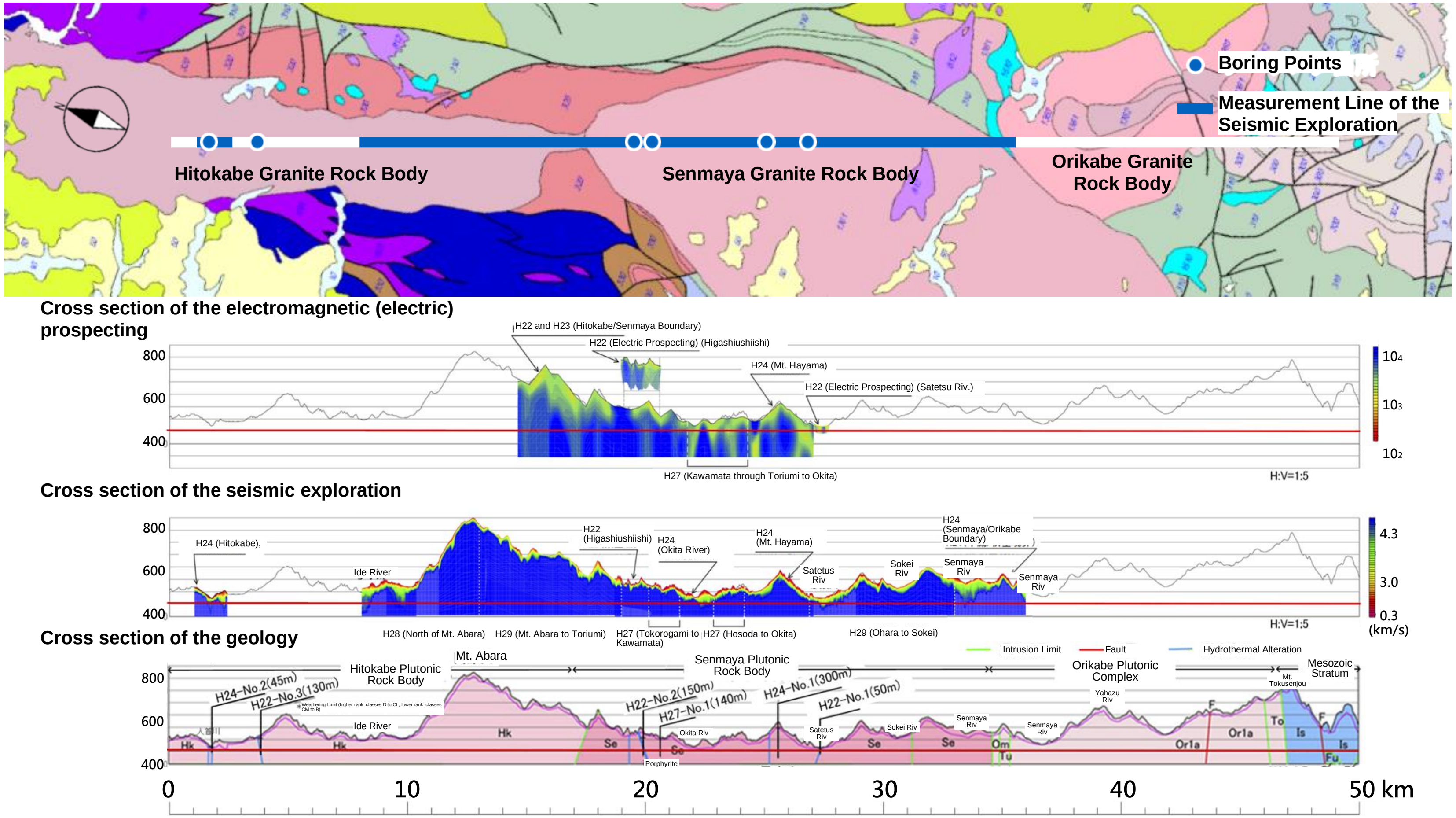


Figure 4.1. Outline of the results of the geological surveys.

4.2 Outline of the results of the geological surveys

Electromagnetic (electric) prospecting

The result of the electromagnetic (electric) prospecting in Figure 4.1 shows that electric resistivities for a depth of 100 m or more were generally as high as 10^4 Ω·m or more while some of them were 10^2 to 10^3 Ω·m locally. Though places with low electric resistivities are suspected to have fissures containing water, rock mass having a few fissures can be expected to extend in the almost whole area.

Seismic exploration

The result of the seismic exploration in Figure 4.1 shows that the elastic wave speeds (P wave speeds) were 4 to 5 km/s, which is very high, at points other than near the ground surface. As described in Chapter 6, the Tohoku ILC Civil Engineering Plan has determined the planned altitude of the ILC tunnel to be 110 m. Figure 4.2 shows elastic wave speeds at altitudes of 100 m and 120 m. The speeds were generally 5 km/s or more near the planned altitude of the ILC tunnel. Though the accuracy of seismic exploration may be problematic near a distance of 5 km due to a large overburden, elastic wave speeds can also be expected to be 5 km or more near an altitude of 110 m because the speeds were over 4 km/s even near the ground surface. On the other hand, the elastic wave speeds on the cross points of rivers with a small overburden were approximately 4 km/s or lower. The cross points of Okita River and Satetsu River had speeds of 3 km/s and 2 km/s, respectively.

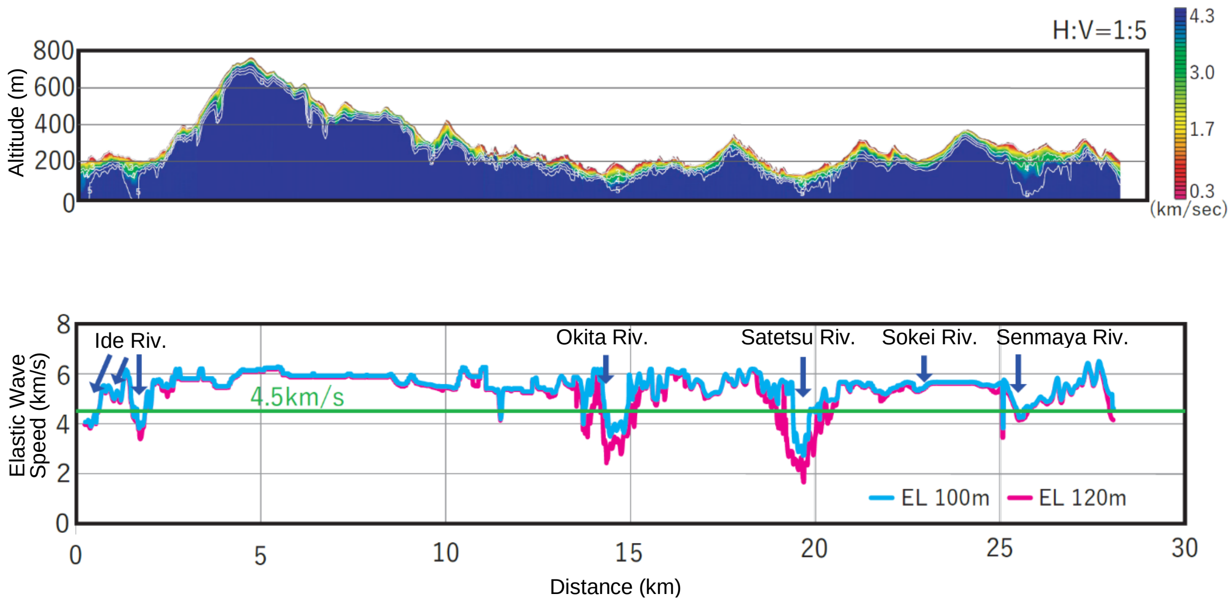


Figure 4.2. Elastic wave speeds (P wave speeds).

Boring surveys

In a boring survey, we obtained the overall length of a core and conducted various kinds of well logging using a borehole.

Figure 4.3 shows pictures and the RQD values of the cores near an altitude of 110 m that is planned for the accelerator tunnel. Granite rocks are shown to be widely distributed at the tunnel altitude regardless of the altitude of the ground surface. In addition, since the RQD values are generally 80% or more for the borings other than the H24-2 and H22-1’ borings, rock mass with a few fissures are supposed to be distributed.

The H24-2 and H22-1/-1’ borings were performed in the riverbanks of Hitokabe River and Satetsu River with the aim of investigating the properties and conditions of surface soil and the weathering conditions of granites in zones with thin overburden. The cores obtained in these two borings were disintegrated into granite sand by finger pressure. The place where the H22-1’ boring had been conducted was assessed as a low specific resistance zone by electric prospecting. On the other hand, the H22-1 boring conducted at a place about 100 m away from the H22-1’ boring resulted in very good quality granite. Thus, the cross zones of the rivers are supposed to have some places locally affected by strong weathering or hydrothermal alteration.

Borehole camera

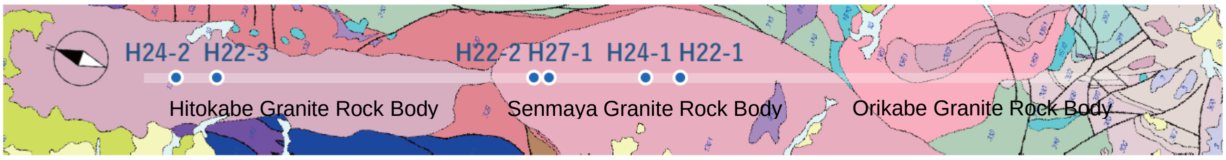
We observed the inner walls of the boreholes with a borehole camera.

The borehole camera observation allowed us to read various fissures mainly consisting of joints. Figure 4.4 shows the frequency distribution of fissures that we observed. The Senmaya granite rock body was shown to have many fissures with a low angle inclination close to the horizontal. The Hitokabe granite rock body was shown to have many fissures inclined in the ENE-WSW strike direction to the south. Figure 4.4 also shows the axial direction of the ILC tunnel and the major axis of the detector hall in the facility location plan described in Chapter 6.

Measurement of initial stress

Using a borehole (H24-1) near the top of Mt. Hayama, we measured initial stress of the ground by a hydraulic fracturing method. Table 4.2 and Figure 4.5 show the result of the measurement.

Since the initial stress obtained by the measurement is the same as those measured in Japan, the site is supposed to have non-special initial stress state.

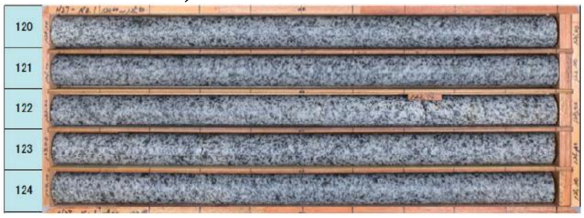


H24-2 (Hitokabe granite rock body, a borehole at an altitude of 146 m)

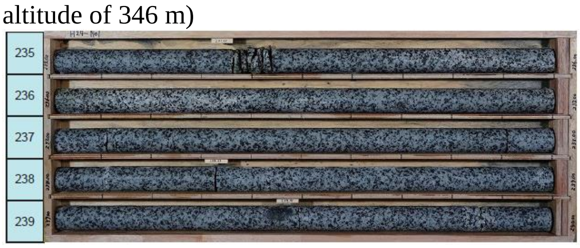
H22-3 (Hitokabe granite rock body, a borehole at an altitude of 210 m)

H22-2 (Senmaya granite rock body, a borehole at an altitude of 214 m)

H27-1 (Senmaya granite rock body, a borehole at an altitude of 231 m)



H24-1 (Senmaya granite rock body, a borehole at an altitude of 346 m)



H22-1 (Senmaya granite rock body, a borehole at an altitude of 128 m)

H22-1’ (Senmaya granite rock body, a borehole at an altitude of 130 m)

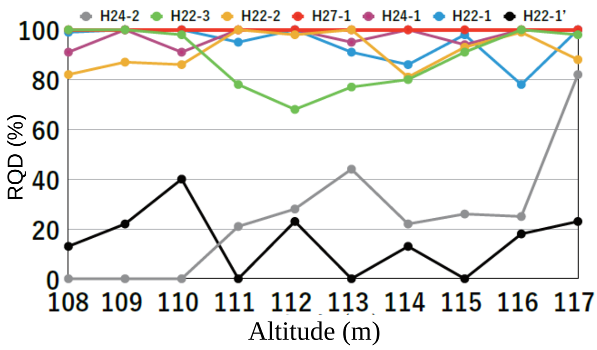
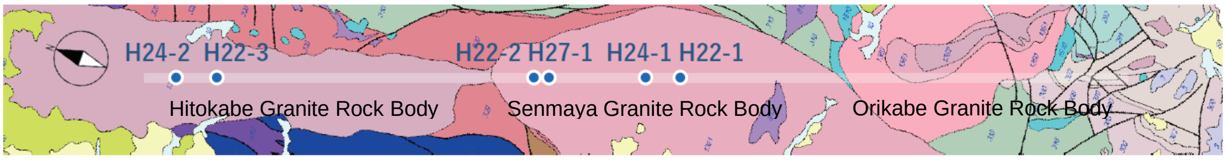
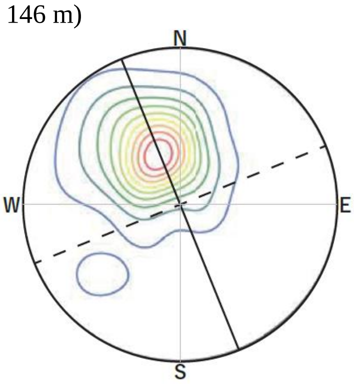


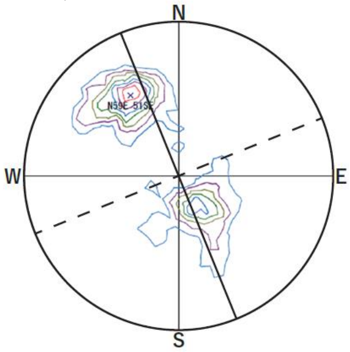
Figure 4.3. Pictures and the RQD of the cores near an altitude of 110 m (the numbers added to the core pictures indicate depths (m)).



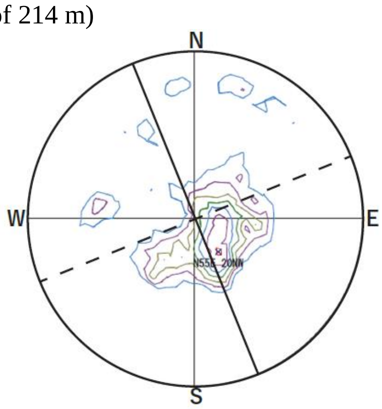
H24-2 (Hitokabe granite rock body, a borehole at an altitude of 146 m)



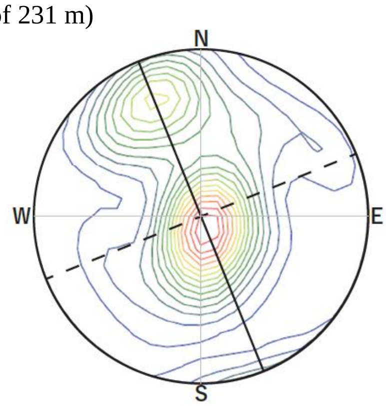
H22-3 (Hitokabe granite rock body, a borehole at an altitude of 210 m)



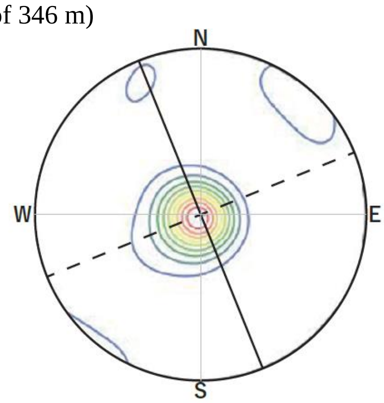
H22-2 (Senmaya granite rock body, a borehole at an altitude of 214 m)



H27-1 (Senmaya granite rock body, a borehole at an altitude of 231 m)



H24-1 (Senmaya granite rock body, a borehole at an altitude of 346 m)



H22-1 (Senmaya granite rock body, a borehole at an altitude of 128 m)

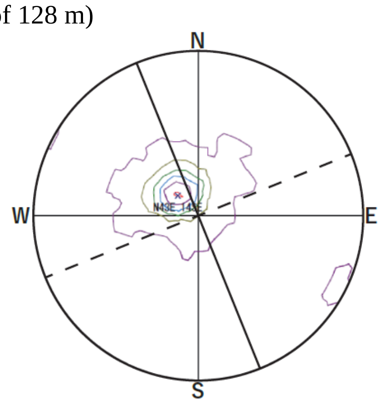


Figure 4.4. Frequency distributions of fissures observed with a borehole camera. The Schmidt net (equal-area projection) is used on a stereonet to express a pole projection chart of a lower hemisphere. Solid lines represent the axial direction of the ILC tunnel, and broken lines represent the main axial direction of the detector hall.

Table 4.2. Result of the measurement of initial stress

Depth (m)	Altitude (m)	Maximum principal stress σHmax (MPa)	Minimum principal stress σHmin (MPa)	Vertical stress[1] σz (MPa)	Maximum lateral stress ratio σHmax/σz	Direction of σHmax (degree)
200.1	145.9	9.49	5.11	5.36	1.77	N44-86W
210.7	135.3	9.68	6.17	5.64	1.71	N54-77W
230.0	116.0	10.72	5.75	6.16	1.74	N14-39W
247.3	98.7	11.31	5.83	6.62	1.71	N21-35W
261.7	84.3	9.45	5.34	7.01	1.35	N32W
296.5	49.5	10.59	6.10	7.94	1.33	N45W

[1] The density of a rock mass is assumed to be 2.73 g/cm^3.

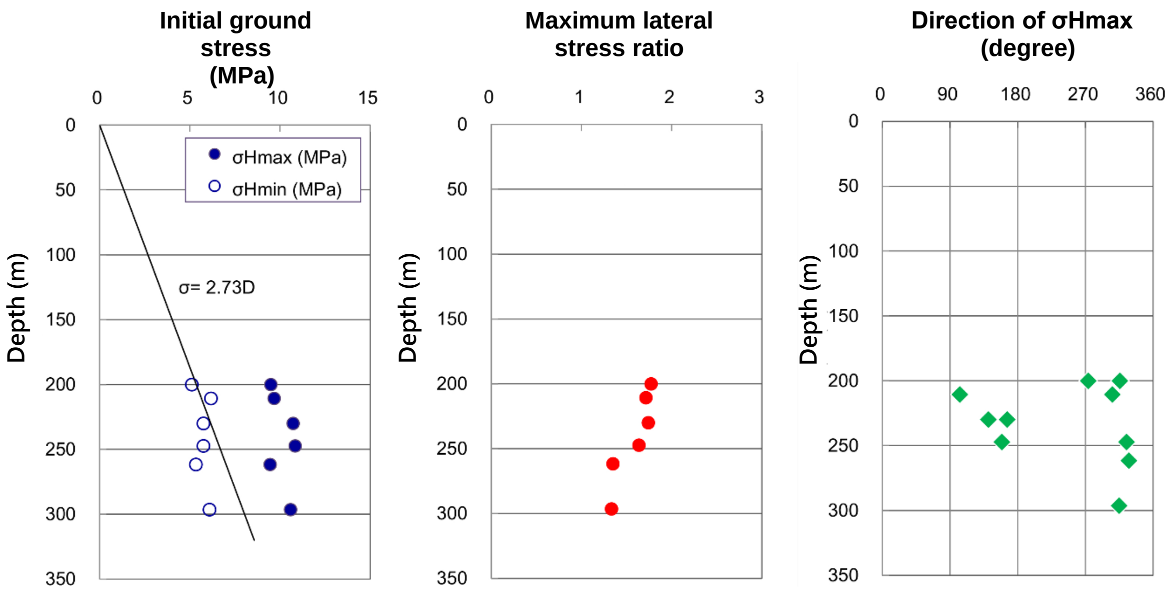


Figure 4.5. Result of the measurement of initial stress.

Laboratory rock specimen tests

We conducted laboratory rock specimen tests for grasping the properties of the granite. For places where the intrusion of porphyrite was shown, we also conducted tests for porphyrite. Table 4.3 shows the results of these tests.

From the measurement, granite rocks at an altitude of 120 m or less were shown to have a uniaxial compressive strength of 129 MPa or more and a splitting tensile strength of 7 MPa or more. Intruding porphyrites were also shown to have almost the same uniaxial compressive strength as the granite. Ultrasonic speeds are consistent with the speeds obtained by the seismic exploration.

Table 4.3. Results of the Laboratory rock specimen tests

Borehole		Altitude of a sample (m)	Type	Uniaxial compressive strength (MPa)	splitting tensile strength (MPa)	Ultrasonic speed	
						V_P (m/s)	V_S (m/s)
Hitokabe granite rock body	H24-2	129.3 – 130.3	Granite	141	11.70	6,290	3,190
	H22-3	95.1 – 95.4	Porphyrite	121	—	5,680	3,400
	H22-3	86.0 – 86.3	Granite	194	—	5,670	3,400
	H22-3	81.5 – 85.8	Granite	166	—	5,780	3,500
Senmaya granite rock body	H22-2	129.0 – 129.3	Porphyrite	253	—	5,770	3,350
	H22-2	87.7 – 88.0	Granite	161	—	5,430	3,240
	H22-2	82.0 – 82.3	Granite	129	—	5,310	3,190
	H24-1	145.7 – 146.0	Granite	90	8.51	6,220	3,170
	H24-1	135.2 – 135.5	Granite	76	8.69	6,230	3,160
	H24-1	115.9 – 116.2	Granite	153	8.74	6,270	3,180
	H24-1	98.6 – 98.9	Granite	142	10.00	6,040	3,140
	H24-1	84.1 – 84.4	Granite	152	7.31	6,100	3,130
	H24-1	49.4 – 49.7	Granite	158	9.28	5,850	3,040
	H22-1	89.4 – 89.7	Granite	176	—	5,240	3,720
	H22-1	87.4 – 88.8	Granite	169	—	5,110	3,180
	H22-1	83.3 – 83.7	Granite	176	—	4,880	2,940

Fluorescence X-ray analysis

The cores obtained by the boring surveys had some friable and/or discolored regions locally. Since these regions are suspected to be affected by mineralization/alteration, we obtained samples from friable/discolored regions to conduct fluorescence X-ray analysis. Table 4.4 shows the results of the analysis. These results of the fluorescence X-ray analysis do not show any signs of mineralization or other effects; thus, the site is supposed to have granite with natural conditions.

Table 4.4. Results of the fluorescence X-ray analysis

		Hitokabe	Hitokabe	Senmaya	Senmaya	Senmaya	Senmaya	Senmaya	Senmaya	Senmaya	Senmaya
Na_2O	%	3.31	4.13	3.39	3.67	3.79	3.77	1.94	0.67	2.18	3.68
MgO	%	1.73	0.84	2.03	2.15	2.05	2.77	1.53	2.07	1.89	2.56
Al_2O_3	%	15.1	17.4	17.5	16.0	16.0	16.1	18.0	20.3	17.0	16.6
SiO_2	%	68.3	75.2	68.0	70.2	67.3	67.2	67.0	63.9	67.4	67.0
P_2O_5	%	0.138	0.075	0.094	0.151	0.108	0.126	0.091	0.099	0.125	0.102
S	%	-0.029	-0.026	-0.026	-0.021	-0.024	-0.026	-0.026	-0.022	-0.022	-0.025
K_2O	%	2.65	2.20	1.57	2.21	1.94	1.82	1.12	1.27	1.28	1.43
CaO	%	4.40	4.45	5.49	4.94	4.97	5.52	7.49	7.94	6.49	5.70
TiO_2	%	0.49	0.22	0.45	0.47	0.46	0.55	0.37	0.38	0.44	0.48
MnO	%	0.08	0.03	0.07	0.08	0.07	0.09	0.09	0.05	0.07	0.09
Fe_2O_3	%	4.52	2.08	4.28	4.33	4.59	5.25	3.95	3.90	4.70	4.96
V	ppm	78.6	43.8	85.2	84.5	89.2	111	94.3	101	102	103
Cr	ppm	15.9	9.36	16.3	15.3	16.7	41.9	34.7	42.3	37.9	37.6
Co	ppm	11.5	4.60	10.8	11.3	12.4	14.3	9.24	8.96	12.1	13.4
Ni	ppm	12.7	9.29	14.9	14.0	15.3	21.1	17.6	15.4	18.1	20.3
Cu	ppm	10.8	9.76	16.8	13.8	15.8	15.9	26.2	56.4	58.8	24.4
Zn	ppm	61.2	37.0	55.3	59.0	58.1	64.9	48.3	45.7	53.7	61.2
As	ppm	1.51	1.83	3.18	1.87	0.67	0.50	0.93	0.61	0.51	0.97
Rb	ppm	79.2	59.4	33.5	63.8	52.9	49.4	19.1	21.0	33.3	40.0
Sr	ppm	276	310	310	348	359	382	163	106	200	380
Y	ppm	10.4	3.33	10.6	14.4	12.3	13.9	9.80	10.2	11.4	15.8
Zr	ppm	102	90.0	101	108	111	107	84.6	93.6	114	133
Nb	ppm	6.60	4.54	3.93	4.78	4.83	4.75	3.96	4.18	4.75	4.81
Sn	ppm	1.17	0.68	0.78	0.76	0.42	0.60	0.91	0.78	0.93	0.82
Sb	ppm	0.49	0.36	0.82	0.60	0.06	0.09	0.50	0.38	0.34	0.11
Cs	ppm	4.54	2.12	1.71	4.06	2.40	1.85	3.22	2.65	4.36	2.32
Ba	ppm	361	347	348	406	363	347	329	248	343	299
La	ppm	23.6	12.4	7.61	4.83	10.7	8.80	7.51	43.9	8.87	10.1
Ce	ppm	37.4	20.8	18.9	13.7	23.1	20.7	17.4	40.8	18.1	24.3
Pr	ppm	2.09	0.80	0.80	0.80	1.54	1.61	0.97	4.24	1.78	4.28
Nd	ppm	10.7	5.54	9.07	8.23	10.3	10.7	7.63	17.7	9.03	12.2
Pb	ppm	14.2	15.2	9.10	11.2	9.52	8.48	9.21	8.95	8.05	8.64
Th	ppm	10.8	5.62	3.34	5.00	3.88	2.85	3.28	3.70	2.42	3.22
Cd	ppm	-0.05	0.14	0.08	-0.10	-0.07	0.10	-0.02	0.14	-0.02	-0.04
U	ppm	3.76	3.68	3.24	4.48	3.52	3.30	1.18	3.23	1.62	3.25

Estimation of the constant groundwater inflow rate

We estimated the constant inflow rate of groundwater into the tunnel based on the topography, tunnel route, and specific discharge of base flow (river flowrate during dry periods divided by the catchment area) by means of the “GIS-based Tunnel Groundwater Inflow-Rate Prediction System"[7] based on the "Hydrogeological Approach"[5,6] developed by the Railway Technical Research Institute (a public interest incorporated foundation). Digital Elevation Model (DEM) of the Fundamental Geospatial Data from the Geospatial Information Authority of Japan was used for the topography modeling, and the data from the Water Information System of the Ministry of Land, Infrastructure, Transport and Tourism (MLIT) was used for the river flow rates of the three river systems (Hitokabe River, Satetsu River, and Senmaya River). As a result, the average groundwater inflow rate along the entire tunnel route was estimated to be 0.8 m3/min/km.

Part III

Tohoku ILC Civil Engineering Plan

5 Circumstances and Conditions for Settling the Project

In the Tohoku Region, the proposed place for ILC location, we have considered various details of the construction of the ILC in the Kitakami Site on the basis of the Technical Design Report (TDR)[1, 2] established by the Global Design Effort.

We entrusted investigation work to the Tohoku Branch of the Japanese Geotechnical Society (a public interest incorporated association) from the 2014 to 2017 fiscal years to examine the details of components such as the ILC accelerator tunnel, detector hall, access tunnels, and the positions of pitheads. Since the 2016 fiscal year, we have been examining them in cooperation with the High Energy Accelerator Research Organization. In the examination, we investigated the results of geological surveys, ambient environment conditions, and other situations, and then selected a reasonable location for the underground facility according to technical and economical evaluation. We also checked the topography of the site and the positions of structures and selected the positions of the pitheads of the access tunnels through on-site investigation. In addition, we considered the construction plan and estimated the processes and costs of the construction.

The Global Design Effort still updated the design during the examination conducted by the above-mentioned entrusted Japanese Geotechnical Society. The main updates of the design were changes in the cross section of a main linear accelerator tunnel (its width was changed to 11 m, to 9 m, and then to 9.5 m in the 2015 and 2016 fiscal years) and in the length at the start of experiments (it was changed from approximately 30 km to approximately 20 km in the 2016 fiscal year).

Because of the circumstances mentioned above, the Tohoku ILC Civil Engineering Plan described from the next chapter is settled under the following conditions.

Construction plan: In accordance with the Civil Engineering Estimation Standards (the 2014 fiscal year edition) of the Ministry of Land, Infrastructure, Transport, and Tourism.

- The plan was in accordance with the above standards even for design changes conducted during the settlement of the plan.
  - For changes in the cross section of a main linear accelerator tunnel, widths of 11 m and 9 m were examined in accordance with the above standards. For the change in width to 9.5 m, the result of the examination for a width of 9 m was converted on the basis of a cross section for the evaluation of costs and excavation processes (per month); for the evaluation of roadbed processes, it was converted on the basis of the tunnel width.
  - For the change in length, unnecessary content due to extension/shortening was able to be excluded.
- When a cross section is in a state of unconformity to the above standards due to its large size, the above standards were applied by setting the cycle time of construction.
- The plan was in accordance with the Technical standards for road tunnels of the Japan Road Association that is equal to the above standards as necessary.
- Shafts were in accordance with the Standard Design Specifications Vol. 3 Tunnels of the East, Central, and West Nippon Expressway Company and the Road Tunnel Technical Standards of the Japan Road Association.

Equipment ownership costs: In accordance with the Table of Construction Equipment Ownership Costs (the 2014 fiscal year edition).

Material unit costs: In accordance with the Construction Material Prices List (August, the 2014 fiscal year) and the Estimation Data (August, the 2014 fiscal year).

Labor unit cost: In accordance with the Unit Labor Cost for Public Works (the 2014 fiscal year).

6 Facility Location

6.1 Location plan

6.1.1 Concepts of the location plan

The ILC facility location plan was established according to the following concepts.

Containing the whole of the ILC accelerators in the granite rock bodies

To minimize risks for tunnel excavation crossing geologic boundaries and faults, we aimed to contain the ILC underground facility in the three granite rock bodies (Hitokabe, Senamaya, and Orikabe granite rock bodies).

If the ILC facility is to be contained in the granite rock bodies, the flexibility of routes is limited because the accelerator tunnel passes the “narrowed point” of the granite existing in the contact point between the Hitokabe and Senmaya granite rock bodies. On the other hand, risks for the tunnel excavation are expected to be minimized because the tunnel can pass the boundary zone of the granite rock bodies along the shortest route.

Unlike circular accelerators, the ILC has the very important feature of being able to improve its performance (to reach further higher energy) by extension. Examining the facility location plan, we checked that it will be able to extend up to 50 km in the future.

Determining the altitude of the accelerator tunnel to be as high as possible (determining the overburden of the shallowest point to be about 2D)

A shorter length access tunnel (inclined shaft) connecting the ground surface and underground facility is advantageous in terms of convenience for usage and construction costs. Thus, we determined the altitude of the accelerator tunnel to be as high as possible. On the other hand, to guarantee the stability of the tunnel and avoid the influences of above ground vibration and temperature changes, we decided to ensure overburden with a thickness of approximately 2D (D is a width of 9.5 m of the accelerator tunnel) for the shallowest point.

The cross point of Satetsu River has the lowest ground surface altitude in this site. The altitude of the ILC underground facility is to be determined by the riverbed altitude of the cross point of Satetsu River.

Considering the present land use conditions

Villages are interspersed along the bank of Satetsu River. For the shallowest point (the cross point of Satetus River), we decided to locate the ILC underground facility without interfering regions where many houses gather.

Considering the stability and workability of the detector hall and shafts

To guarantee the stability of the detector hall, we planned to ensure intact rocks with a thickness of 2D (D is a height of 42 m of the detector hall) as overburden for the hall.

The ILC underground facility location plan was settled according to the above-mentioned concepts. If it is decided that the accelerator tunnel is to pass the Hitokabe granite rock body that is long in the north-south direction and the “narrowed point,” its location in the east-west direction and angle of direction are almost determined. In addition, the altitude of the floor of the tunnel is determined to be 110 m above sea level according to the altitude of the shallowest point (the cross point of Satetsu River).

Assuming that the thickness of a weathered layer is 30 m with reference to the result of the geological surveys, the detector hall should be constructed at a ground surface altitude of approximately 250 m.

The location in the north-south direction of the ILC facility depends on the position of the detector hall. Under the condition that both ground surface altitude is approximately 250 m and the above-ground facilities necessary for the operation of the ILC can be constructed, we examined the following two facility location plans: one is that the detector hall is placed to the south of Okita River, and the other is that the hall is placed to the north of Okita River. The comparison between these two plans shows that the former has advantages in terms of the construction period and costs of public works, while the latter features a highly flexible design. From this point forward, we describe the former, the “facility location plan to the south of Okita River.” Note that the granite rock bodies can be excavated in any place other than the southern end of the accelerator tunnel when the tunnel will be extended to 50 km in the future.

6.1.2 ILC facility location plan

Figures 6.1[3] and 6.2[4] show the ILC facility location plan. As shown in Figures 3.1 and 6.1, the surrounding area of the ILC facility tends to have a higher altitude on its eastern side than its western side. Since the lower altitude of a pithead is advantageous in shortening the length of an access tunnel, the access tunnels are determined to connect to the accelerator tunnel from the western side of the ILC facility. As shown in Figure 6.2, in addition, the ILC facility is contained in the Hitokabe and Senmaya granite rock bodies.

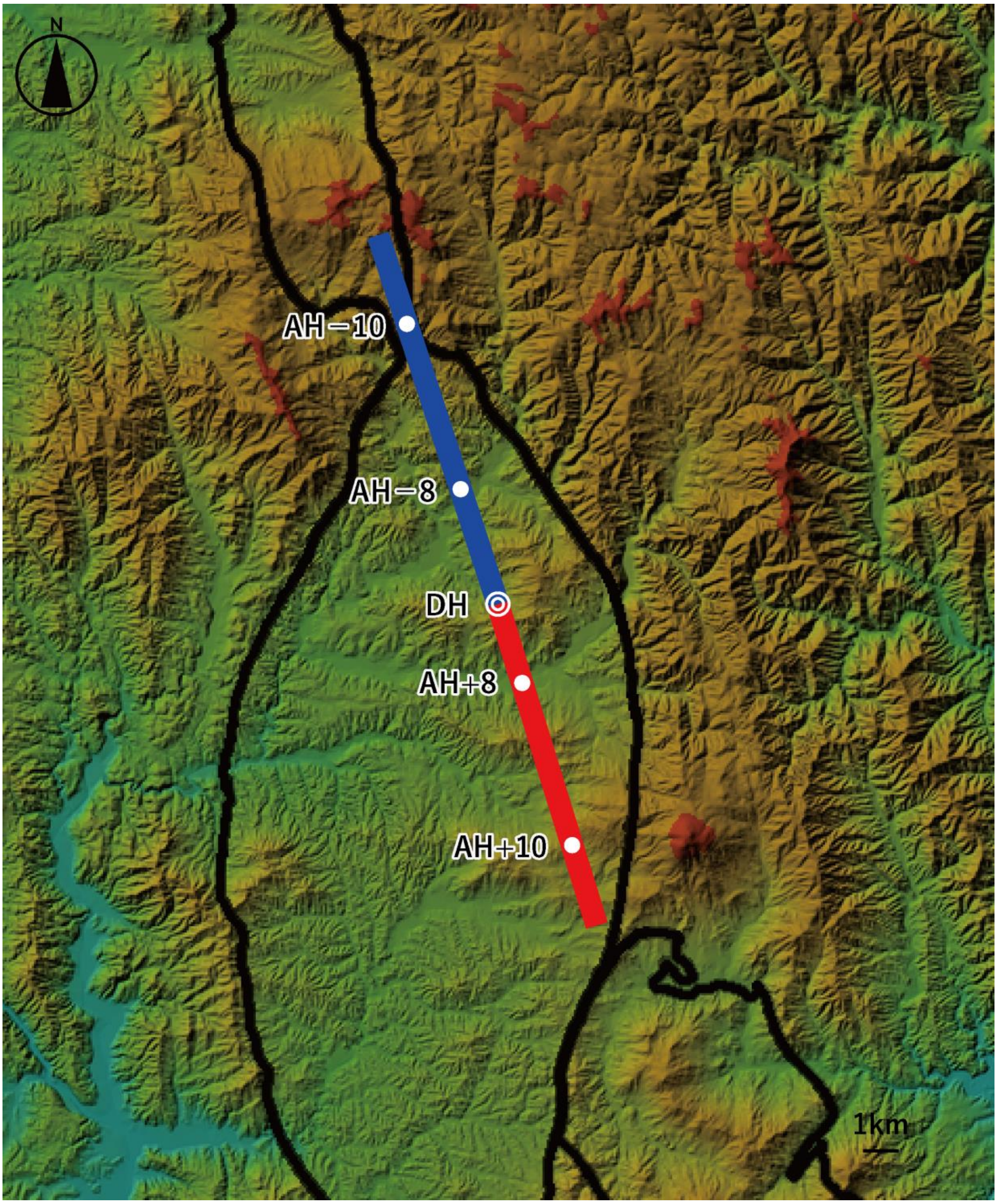


Figure 6.1. A plan view of the topography of the ILC facility location plan (blue line: the main linear accelerator for electrons (a length of 10.9 km), red line: the main linear accelerator for positrons (a length of 9.6 km)).

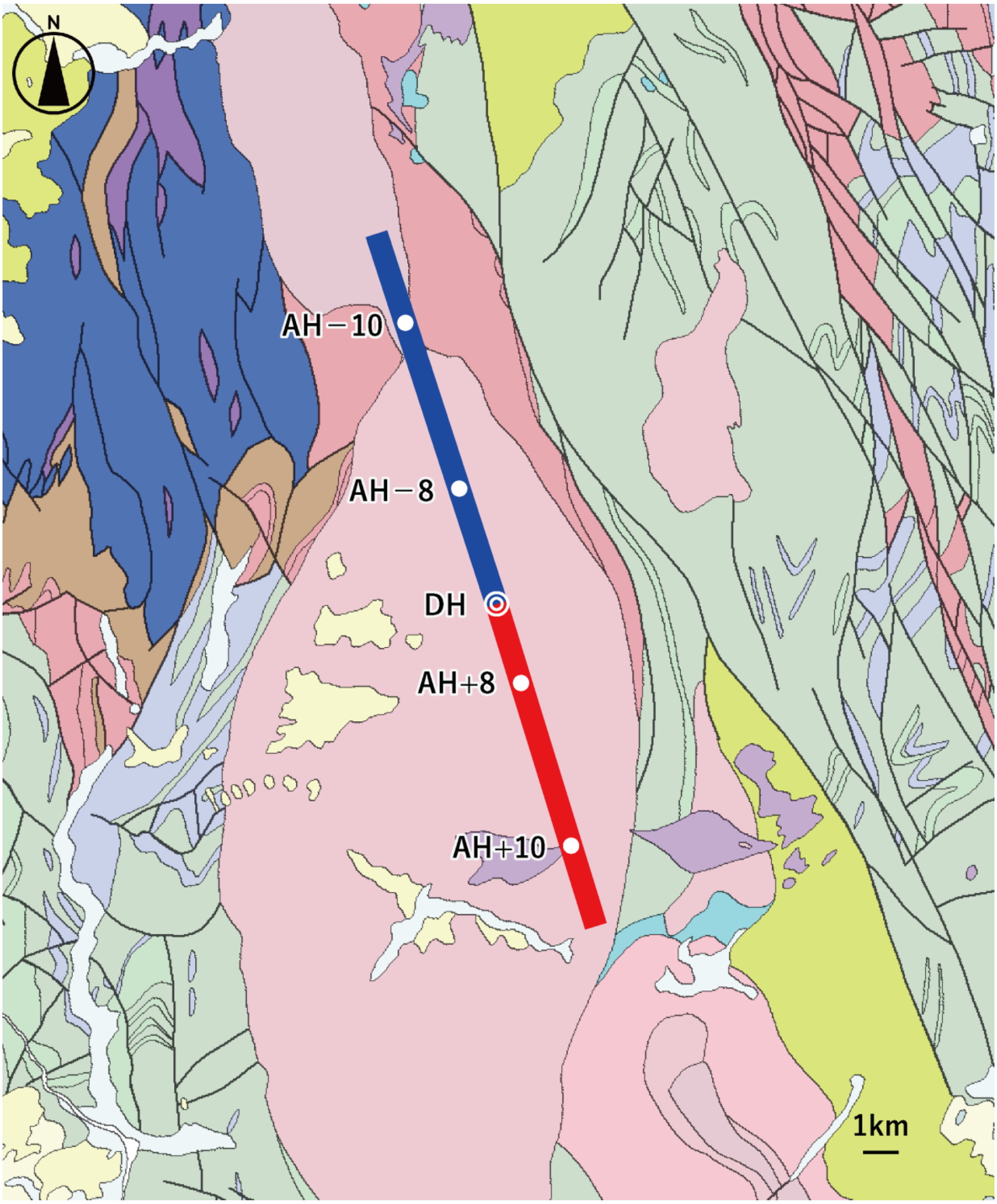


Figure 6.2. A plan view of the geology of the ILC facility location plan (blue line: the main linear accelerator for electrons (a length of 10.9 km), red line: the main linear accelerator for positrons (a length of 9.6 km)).

7 Access Tunnels and Access Hall

7.1 Design specifications

7.1.1 Selection of the positions of pitheads

As shown in Figure 2.7, since the distance of the access halls to the collision point is determined by the design of the accelerator, the connection points between the accelerator tunnel and access tunnels are determined according to the position of the collision point. We narrowed the proposed positions of pitheads under this condition so that the lengths of the access tunnels are as short as possible. In addition, we conducted an on-site investigation to research the proposed positions and the vicinity, and established the positions of pitheads more realistically.

The gradient of an access tunnel was determined to be 10% sloping downward for 90% of the overall length and horizontal for 10%; thus, an access tunnel was determined to extend from the pithead to the boundary of an access hall with an average falling gradient of 9%. The floor of an access hall was determined to be horizontal, and the altitude of the floor was determined to be 110 m which is same as that of the main linear accelerator tunnels.

Table 7.1 shows a list of the specifications of the access tunnels. A yard area in the table is a site area that can be ensured in front of a pithead. The uniform area of 19,000 m^2 means that it can ensure an area (18,200 m^2) necessary for the installation of equipment and other purposes after the completion of the public works. In addition, two access tunnels connecting to the detector hall (AT-DH) and to the damping rings (AT-DR) were planned to have the same pithead and pithead yard (AS-DH and AS-DR are same) and diverge midway.

Table 7.1. List of the specifications of the access tunnels

Entry point (pithead)	Yard area (m^2)	Pithead altitude (m)	Average gradient (%)	Length (m)
AS−10	19,000	250	−9.0	1,707
AS− 8	19,000	170	−9.0	817
AS−DH	19,000	170	−3.1	747
AS−DR			−9.0	817
AS+ 8	19,000	135	−9.0	427
AS+10	19,000	200	−9.0	1,147

7.1.2 Rock mass classes

The access tunnels are to be excavated from the ground surface by passing a weathering zone though a hard rock mass with classes B to CH is assumed for the evaluation of the rock mass at the depth of the main linear accelerator tunnels. Considering weathering progression near to the overburden of 30 m with reference to the results of the on-site investigation and seismic exploration, support patterns of the access tunnels were assumed to be as follows according to the depth of overburden. Tunnel support patterns corresponding to rock mass classes were determined in accordance with the Public Work Estimation Standards of the Ministry of Land, Infrastructure, Transport, and Tourism.

- From the “pithead” to the “overburden of 2D (16 m)”: Pattern pithead-DIII
- From the “overburden of 2D (16 m)” to the “overburden of 30 m” (weathering zone): Pattern DI
- From the “overburden of 30 m” to the “floor altitude same as the main linear accelerator tunnels (110 m)”: Pattern CII
- The “floor altitude same as the main linear accelerator tunnels (110 m)”: Pattern CI

To conduct more detailed design, detailed geological surveys are required for pithead points as necessary.

7.1.3 Dimensions and structures of the access tunnels and access halls

The cross sections of access tunnels and access halls are as follows:

1. Access tunnel: a width of 8.0 m and a height of 7.5 m
2. Access halls (S dome, E dome, and M dome): a width of 14.0 m and a height of 12.0 m
3. Access hall (He dome): a width of 20.0 m and a height of 12.0 m

A tunnel with a width of 8.0 m has difficulty in allowing dump trucks, concrete mixing transport trucks, and crane trucks to change direction. Considering a vehicle can change its direction during construction and can easily pass after the completion, it was decided to have access tunnels with emergency parking zones and reversal pits (with a widening of 2 m and a length of 30 m horizontally) placed at intervals of approximately 300 m.

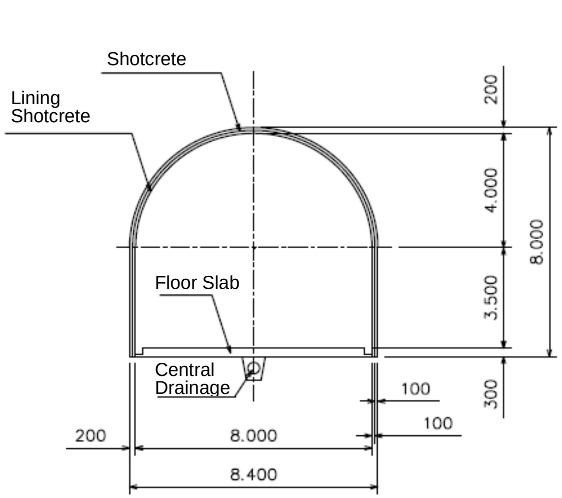
For the access tunnels connecting the detector hall and damping rings, it was decided to widen the finished width of the cross section of the common section (a length of 537 m) to 10.0 m (same as that of an emergency parking zone) because large vehicles are congested in the section from the pithead to the fork.

Access tunnels other than the pithead-DIII section were determined to have single shell structures omitting lining concrete, where shotcrete lining with a thickness of 15 cm was determined, as well as shotcrete for primary support during excavation.

The access halls were determined to have structures with shotcrete lining instead of lining concrete, even though they contain devices. Standard cross-sectional views and support patterns of each rock mass class of the access tunnels are shown as follows: Figures 7.1 to 7.4 show those of the standard section; Figures 7.5 to 7.7 show those of a section for emergency parking zones; and Figure 7.8 shows those of a section for widening. In addition, Figures 7.9 and 7.10 show standard cross-sectional views and support patterns of an He dome and an S/E/M dome of the access halls.

Other outlines of structures are as follows:

- The pithead-DIII section applied an all ground fastening method (AGF method) and was determined to have an invert closing (ring structure) cross section. In addition, lining concrete (with reinforcing iron bars and waterproof sheets) was determined.
- For waterproof work, the pithead-DIII section was determined to have waterproof sheets, and other sections were determined to have only drainage materials because of single shell structures.



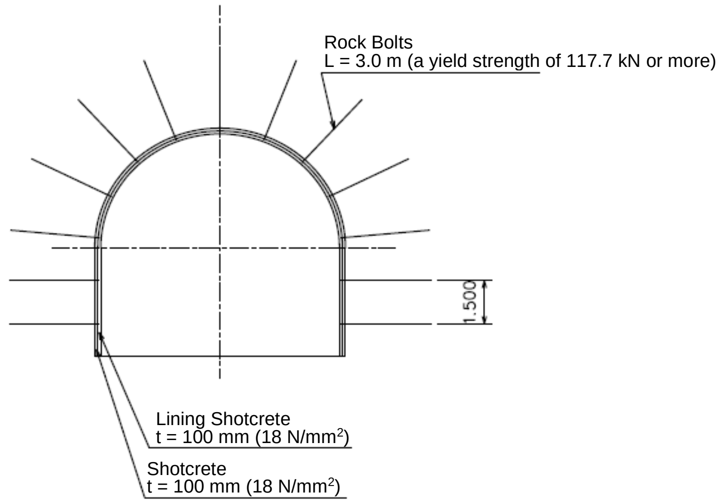


Table of unit quantities

Name	Specification	Unit	Quantity	Remarks
Cross section of excavation (design)		m^2	59.854	Per 1.0 m
Cross section of excavation (pay line)	An outbreak of 22 cm	m^2	64.521	Per 1.0 m
Steel arch support	H-125	units	-	Per 1.5 m
Shotcrete	t =100	m^2	20.555	Per 1.0 m
Rock bolt	L = 3.0 m, a yield strength of 117.7 kN or more	pieces	13	Per 1.5 m
Shot lining	t =100	m^2	20.556	Per 1.0 m
Lining concrete	t =300	m^3	-	Per 1.0 m
Roadbed concrete	t =400	m^3	3.1	Per 1.0 m
Central drainage (drainpipe)	Φ300 (perforated pipe)	m	10.0	Per 10.0 m
Central drainage (excavation)		m^3	6.956	Per 10.0 m
Central drainage (filter)	s -20	m^3	6.016	Per 10.0 m

Table of specifications

Rock bolt			Steel arch support		Shot thickness	Wire net	Shot lining	Lining thickness		Assistance method
Length	Circumferential direction	Length direction	Upper half	Lower half				Arch/side wall	Invert	
3.0	1.5	1.5	-	-	10	-	10	-	-	-

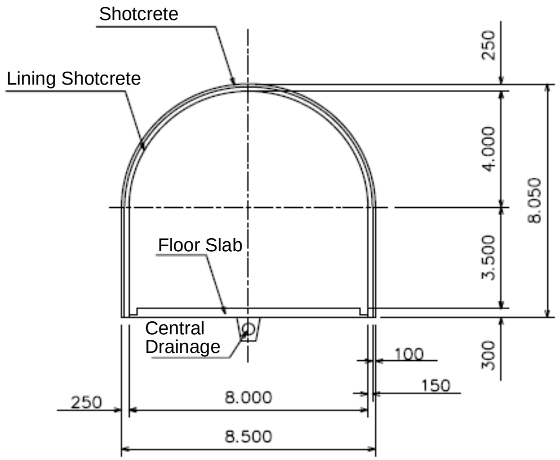
Table of the amount of a shot material

Name	Dimension	Standard	Unit	Quantity	Remarks
Shotcrete	t =100	σ28=18N/mm2	m2/m	20.555	

Table of the quantity of rock bolt materials

Name	Dimension	Standard	Unit	Quantity	Remarks
Rock bolt	L=3,000	A yield strength of 117.7 kN or more	pieces	13	Mortar adheres to all surfaces
Washer	150x150x9	s s 400	pieces	13	
Nut		M24	pieces	13	

Figure 7.1. Standard cross-sectional view (left) and support pattern (right) of the standard section of an access tunnel in the CI section.



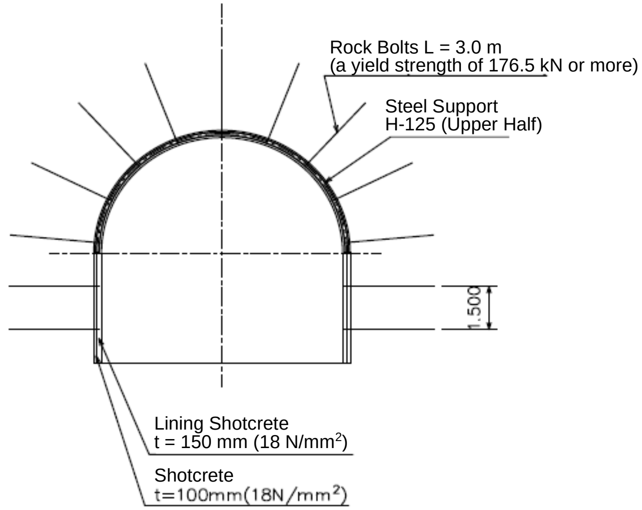


Table of unit quantities

Name	Specification	Unit	Quantity	Remarks
Cross section of excavation (design)		m^2	60.900	Per 1.0 m
Cross section of excavation (pay line)	An outbreak of 20 cm	m^2	65.169	Per 1.0 m
Steel arch support	H-125	units	1 for upper half	Per 1.2 m
Shotcrete	t=100	m^2	20.713	Per 1.0 m
Rock bolt	L = 3.0 m, a yield strength of 17.65 kN or more	pieces	13	Per 1.2 m
Shot lining	t=150	m^2	20.714	Per 1.0 m
Lining concrete	t=300	m^3	-	Per 1.0 m
Roadbed concrete	t=400	m^3	3.1	Per 1.0 m
Central drainage (drainpipe)	Φ300 (perforated pipe)	m	10	Per 10.0 m
Central drainage (excavation)		m^3	6.956	Per 10.0 m
Central drainage (filter)	s-20	m^3	6.016	Per 10.0 m

Table of specifications

Rock bolt			Steel arch support		Shot thickness	Wire net	Shot lining	Lining thickness		Assistance method
Length	Circumferential direction	Length direction	Upper half	Lower half				Arch/side wall	Invert	
3.0	1.5	1.2	H-125	-	10	-	15	-	-	-

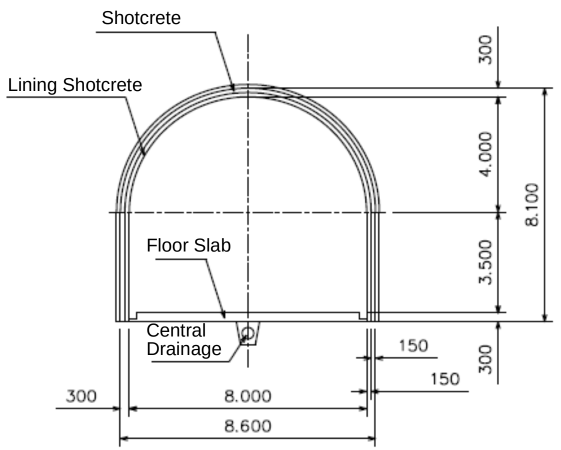
Table of the amount of a shot material

Name	Dimension	Standard	Unit	Quantity	Remarks
Shotcrete	t=100	σ28=18N/mm2	m2/m	20.714	

Table of the quantity of rock bolt materials

Name	Dimension	Standard	Unit	Quantity	Remarks
Rock bolt	L=3,000	A yield strength of 17.65 kN or more	pieces	13	Mortar adheres to all surfaces
Washer	150x150x9	ss400	pieces	13	
Nut		M24	pieces	13	

Figure 7.2. Standard cross-sectional view (left) and support pattern (right) of the standard section of an access tunnel in the CII section.



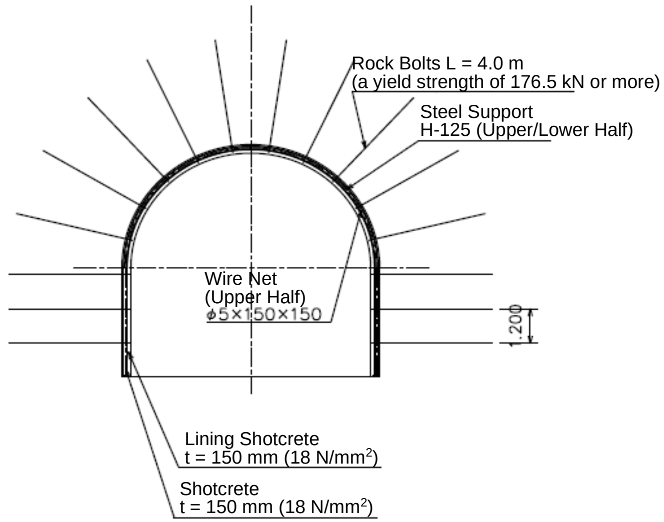


Table of unit quantities

Name	Specification	Unit	Quantity	Remarks
Cross section of excavation (design)		m^2	61.956	Per 1.0 m
Cross section of excavation (pay line)	An outbreak of 17 cm	m^2	65.603	Per 1.0 m
Steel arch support	H-125	units	1 for upper/lower half	Per 1.0 m
Shotcrete	t=150	m^2	20.713	Per 1.0 m
Rock bolt	L = 4.0 m, a yield strength of 17.65 kN or more	pieces	16	Per 1.0 m
Shot lining	t=150	m^2	20.714	Per 1.0 m
Lining concrete	t=300	m^3	-	Per 1.0 m
Roadbed concrete	t=400	m^3	3.1	Per 1.0 m
Central drainage (drainpipe)	Φ300 (perforated pipe)	m	10	Per 10.0 m
Central drainage (excavation)		m^3	6.956	Per 10.0 m
Central drainage (filter)	s-20	m^3	6.016	Per 10.0 m

Table of specifications

Rock bolt			Steel arch support		Shot thickness	Wire net	Shot lining	Lining thickness		Assistance method
Length	Circumferential direction	Length direction	Upper half	Lower half				Arch/side wall	Invert	
4.0	1.2	1.0	H-125	H-125	15	Upper half	15	-	-	-

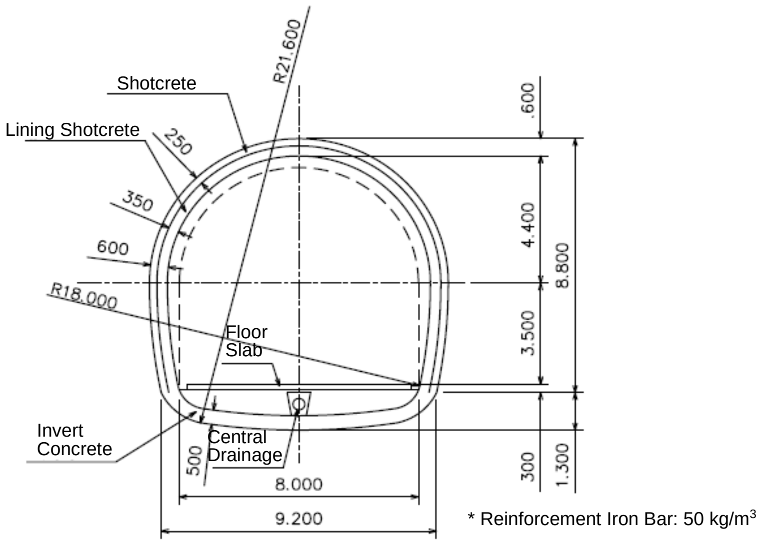
Table of the amount of a shot material

Name	Dimension	Standard	Unit	Quantity	Remarks
Shotcrete	t=150	σ28=18N/mm2	m2/m	20.713	

Table of the quantity of rock bolt materials

Name	Dimension	Standard	Unit	Quantity	Remarks
Rock bolt	L=4,000	A yield strength of 17.65 kN or more	pieces	16	Mortar adheres to all surfaces
Washer	150x150x9	ss400	pieces	16	
Nut		M24	pieces	16	

Figure 7.3. Standard cross-sectional view (left) and support pattern (right) of the standard section of an access tunnel in the DI section.



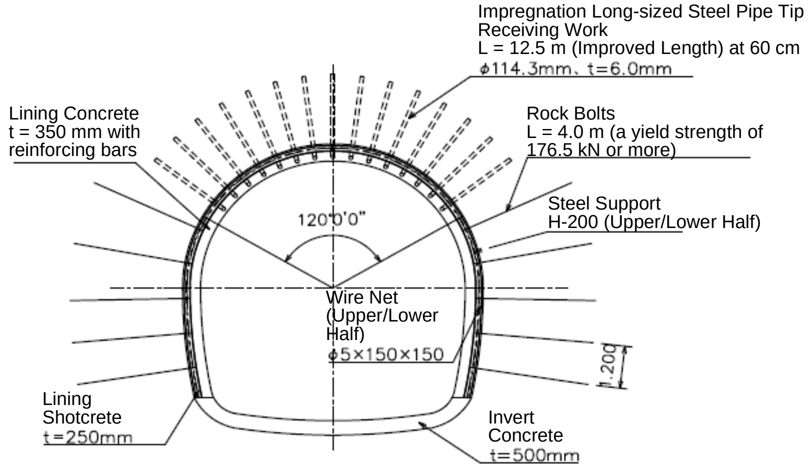


Table of unit quantities

Name	Specification	Unit	Quantity	Remarks
Cross section of excavation (design)		m^2	86.425	Per 1.0 m
Cross section of excavation (pay line)	An outbreak of 17 cm	m^2	90.967	Per 1.0 m
Steel arch support	H-200	units	1 for upper/lower half	Per 1.0 m
Shotcrete	t=250	m^2	23.448	Per 1.0 m
Rock bolt	L = 4.0 m, a yield strength of 17.65 kN or more	pieces	10	Per 1.0 m
Shot lining		m^2	-	Per 1.0 m
Lining concrete	t = 300, invert with t = 500	m^3	15.202	Per 1.0 m
Roadbed concrete	t=400	m^3	3.1	Per 1.0 m
Central drainage (drainpipe)	Φ300 (perforated pipe)	m	10	Per 10.0 m
Central drainage (excavation)		m^3	-	Per 10.0 m
Central drainage (filter)	s-20	m^3	6.016	Per 10.0 m

Table of specifications

Rock bolt			Steel arch support		Shot thickness	Wire net	Shot lining	Lining thickness		Assistance method
Length	Circumferential direction	Length direction	Upper half	Lower half				Arch/side wall	Invert	
4.0	1.2	1.0	H-200	H-200	25	Upper/lower half	-	300	500	AGF

Table of the amount of a shot material

Name	Dimension	Standard	Unit	Quantity	Remarks
Shotcrete	t=250	σ28=18N/mm2	m2/m	23.448	

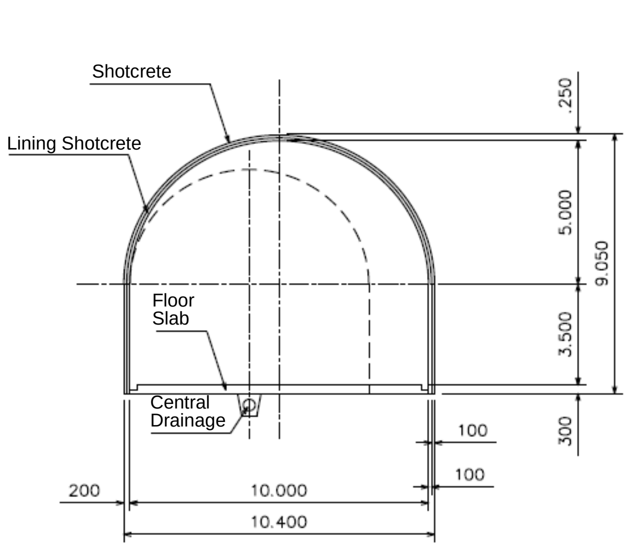
Table of the quantity of rock bolt materials

Name	Dimension	Standard	Unit	Quantity	Remarks
Rock bolt	L=4,000	A yield strength of 17.65 kN or more	pieces	10	Mortar adheres to all surfaces
Washer	150x150x9	ss400	pieces	10	
Nut		M24	pieces	10	

Table of the quantity of an AGF material

Improved length	Circumferential direction	Length direction	Diameter of a steel pipe	Thickness of a steel pipe		Unit	Quantity	Remarks
12.5m	@6cm	@6.0m	φ114.3	t=6.0		pieces	15	Impregnation method

Figure 7.4. Standard cross-sectional view (left) and support pattern (right) of the standard section of an access tunnel in the DIII section.



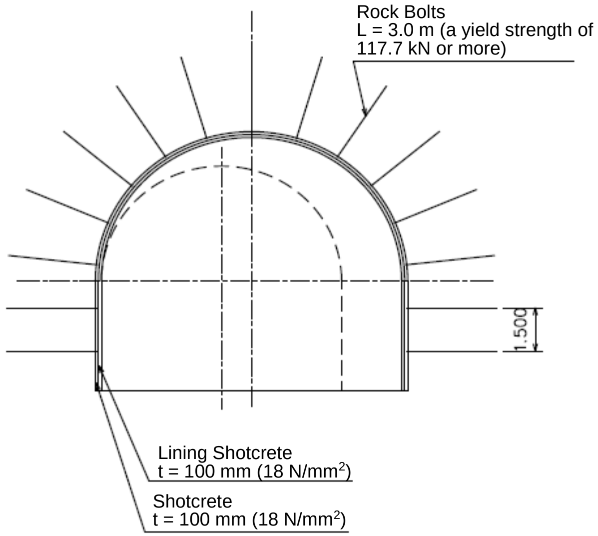


Table of unit quantities

Name	Specification	Unit	Quantity	Remarks
Cross section of excavation (design)		m^2	81.973	Per 1.0 m
Cross section of excavation (pay line)	An outbreak of 22 cm	m^2	87.313	Per 1.0 m
Steel arch support	H-125	units	-	Per 1.5 m
Shotcrete	t=100	m^2	23.614	Per 1.0 m
Rock bolt	L = 3.0 m, a yield strength of 117.7 kN or more	pieces	15	Per 1.5 m
Shot lining	t=100	m^2	23.614	Per 1.0 m
Lining concrete	t=300	m^3	-	Per 1.0 m
Roadbed concrete	t=400	m^3	3.9	Per 1.0 m
Central drainage (drainpipe)	Φ300 (perforated pipe)	m	10	Per 10.0 m
Central drainage (excavation)		m^3	6.956	Per 10.0 m
Central drainage (filter)	s-20	m^3	6.016	Per 10.0 m

Table of specifications

Rock bolt			Steel arch support		Shot thickness	Wire net	Shot lining	Lining thickness		Assistance method
Length	Circumferential direction	Length direction	Upper half	Lower half				Arch/side wall	Invert	
3.0	1.5	1.5	-	-	10	-	10	-	-	-

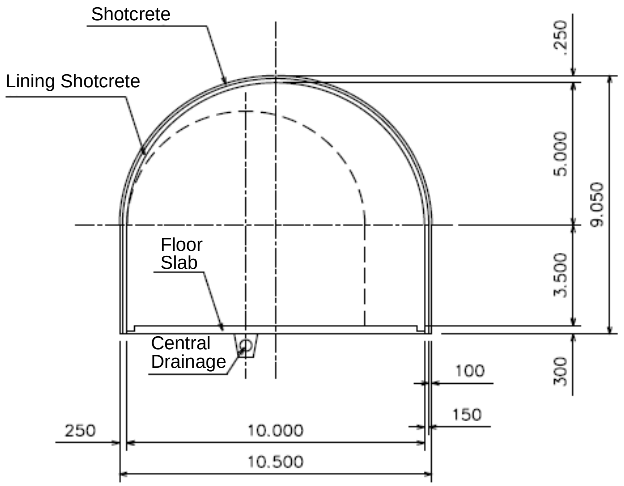
Table of the amount of a shot material

Name	Dimension	Standard	Unit	Quantity	Remarks
Shotcrete	t=100	σ28=18N/mm2	m2/m	23.614	

Table of the quantity of rock bolt materials

Name	Dimension	Standard	Unit	Quantity	Remarks
Rock bolt	L=3,000	A yield strength of 117.7 kN or more	pieces		Mortar adheres to all surfaces
Washer	150x150x9	ss400	pieces		
Nut		M24	pieces		

Figure 7.5. Standard cross-sectional view (left) and support pattern (right) of the emergency parking zone section of an access tunnel and the common section of an access tunnel connecting the detector hall and damping rings in the CI section.



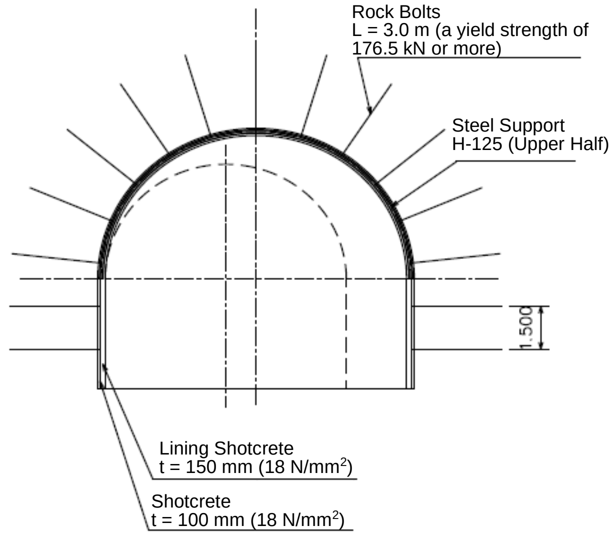


Table of unit quantities

Name	Specification	Unit	Quantity	Remarks
Cross section of excavation (design)		m^2	83.506	Per 1.0 m
Cross section of excavation (pay line)	An outbreak of 20 cm	m^2	88.406	Per 1.0 m
Steel arch support	H-125	units	1 for upper half	Per 1.2 m
Shotcrete	t=100	m^2	23.866	Per 1.0 m
Rock bolt	L = 3.0 m, a yield strength of 17.65 kN or more	pieces	15	Per 1.2 m
Shot lining	t=100	m^2	35.799	Per 1.0 m
Lining concrete	t=300	m^3	-	Per 1.0 m
Roadbed concrete	t=400	m^3	3.9	Per 1.0 m
Central drainage (drainpipe)	Φ300 (perforated pipe)	m	10	Per 10.0 m
Central drainage (excavation)		m^3	6.956	Per 10.0 m
Central drainage (filter)	s-20	m^3	6.016	Per 10.0 m

Table of specifications

Rock bolt			Steel arch support		Shot thickness	Wire net	Shot lining	Lining thickness		Assistance method
Length	Circumferential direction	Length direction	Upper half	Lower half				Arch/side wall	Invert	
3.0	1.5	1.2	H-125	-	10	-	15	-	-	-

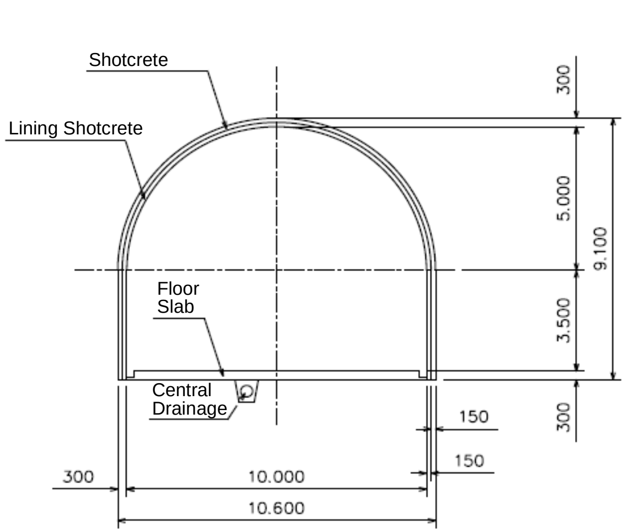
Table of the amount of a shot material

Name	Dimension	Standard	Unit	Quantity	Remarks
Shotcrete	t=100	σ28=18N/mm2	m2/m	23.866	

Table of the quantity of rock bolt materials

Name	Dimension	Standard	Unit	Quantity	Remarks
Rock bolt	L=3,000	A yield strength of 17.65 kN or more	pieces	15	Mortar adheres to all surfaces
Washer	150x150x9	ss400	pieces	15	
Nut		M24	pieces	15	

Figure 7.6. Standard cross-sectional view (left) and support pattern (right) of the emergency parking zone section of an access tunnel and the common section of an access tunnel connecting the detector hall and damping rings in the CII section.



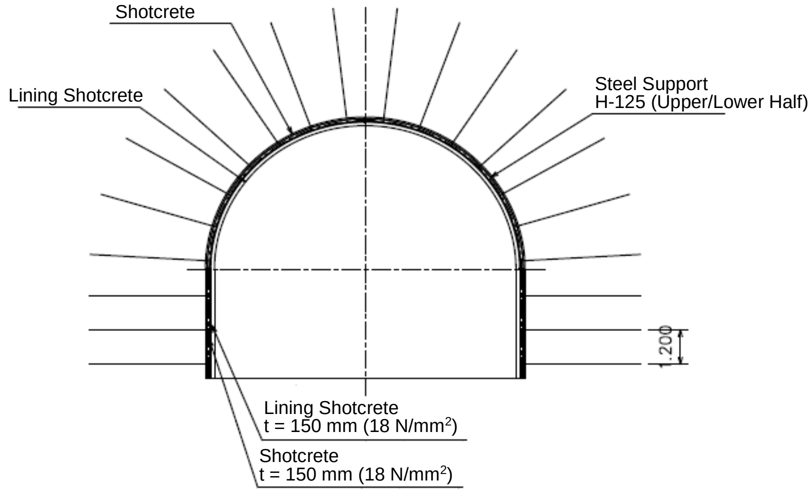


Table of unit quantities

Name	Specification	Unit	Quantity	Remarks
Cross section of excavation (design)		m2	84.404	Per 1.0 m
Cross section of excavation (pay line)	An outbreak of 17 cm	m2	89.815	Per 1.0 m
Steel arch support	H-125	units	1 for upper/lower half	Per 1.0 m
Shotcrete	t=150	m2	23.866	Per 1.0 m
Rock bolt	L = 4.0 m, a yield strength of 17.65 kN or more	pieces	20	Per 1.0 m
Shot lining	t=150	m2	23.866	Per 1.0 m
Lining concrete	t=300	m3	-	Per 1.0 m
Roadbed concrete	t=400	m3	3.9	Per 1.0 m
Central drainage (drainpipe)	Φ300 (perforated pipe)	m	10	Per 10.0 m
Central drainage (excavation)		m3	6.956	Per 10.0 m
Central drainage (filter)	s-20	m3	6.016	Per 10.0 m

Table of specifications

Rock bolt			Steel arch support		Shot thickness	Wire net	Shot lining	Lining thickness		Assistance method
Length	Circumferential direction	Length direction	Upper half	Lower half				Arch/side wall	Invert	
4.0	1.2	1.0	H-125	H-125	15	Upper half	15	-	-	-

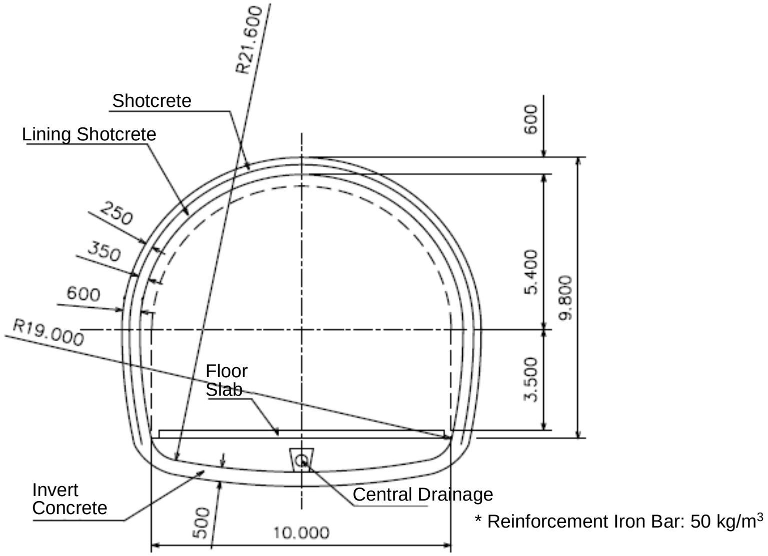
Table of the amount of a shot material

Name	Dimension	Standard	Unit	Quantity	Remarks
Shotcrete	t=150	σ28=18N/mm2	m2/m	23.866	

Table of the quantity of rock bolt materials

Name	Dimension	Standard	Unit	Quantity	Remarks
Rock bolt	L=4,000	A yield strength of 17.65 kN or more	pieces	20	Mortar adheres to all surfaces
Washer	150x150x9	ss400	pieces	20	
Nut		M24	pieces	20	

Figure 7.7. Standard cross-sectional view (left) and support pattern (right) of the emergency parking zone section of an access tunnel and the common section of an access tunnel connecting the detector hall and damping rings in the DI section.



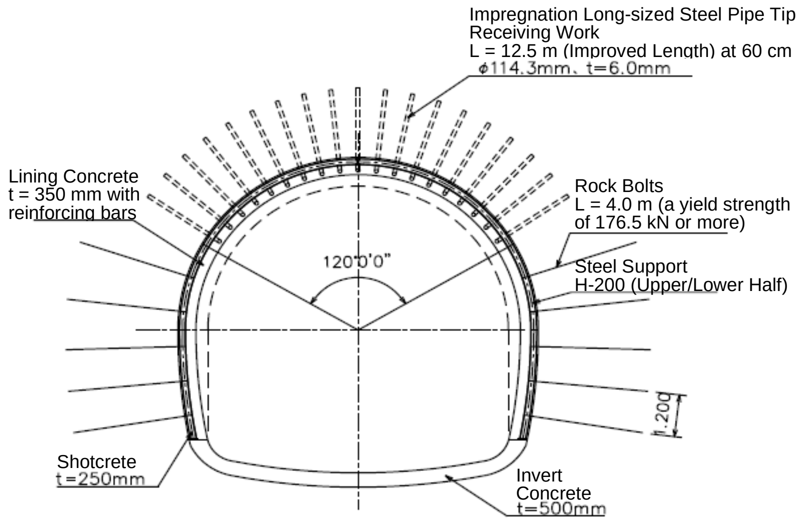


Table of unit quantities

Name	Specification	Unit	Quantity	Remarks
Cross section of excavation (design)		m^2	154.438	Per 1.0 m
Cross section of excavation (pay line)	An outbreak of 17 cm	m^2	164.337	Per 1.0 m
Steel arch support	H-200	units	1 for upper/lower half	Per 1.5 m
Shotcrete	t=250	m^2	26.137	Per 1.0 m
Rock bolt	L = 4.0 m, a yield strength of 17.65 kN or more	pieces	10	Per 1.5 m
Shot lining		m^2	-	Per 1.0 m
Lining concrete	t = 300, invert with t = 500	m^3	16.945	Per 1.0 m
Roadbed concrete	t=400	m^3	3.9	Per 1.0 m
Central drainage (drainpipe)	Φ300 (perforated pipe)	m	10	Per 10.0 m
Central drainage (excavation)		m^3	-	Per 10.0 m
Central drainage (filter)	s-20	m^3	6.016	Per 10.0 m

Table of specifications

Rock bolt			Steel arch support		hot thickness	Wire net	Shot lining	Lining thickness		Assistance method
Length	Circumferential direction	Length direction	Upper half	Lower half				Arch/side wall	Invert	
4.0	1.2	1.0	H-200	H-200	250	Upper/lower half	-	300	500	AGF

Table of the amount of a shot material

Name	Dimension	Standard	Unit	Quantity	Remarks
Shotcrete	t=250	σ28=18N/mm2	m2/m	26.137	

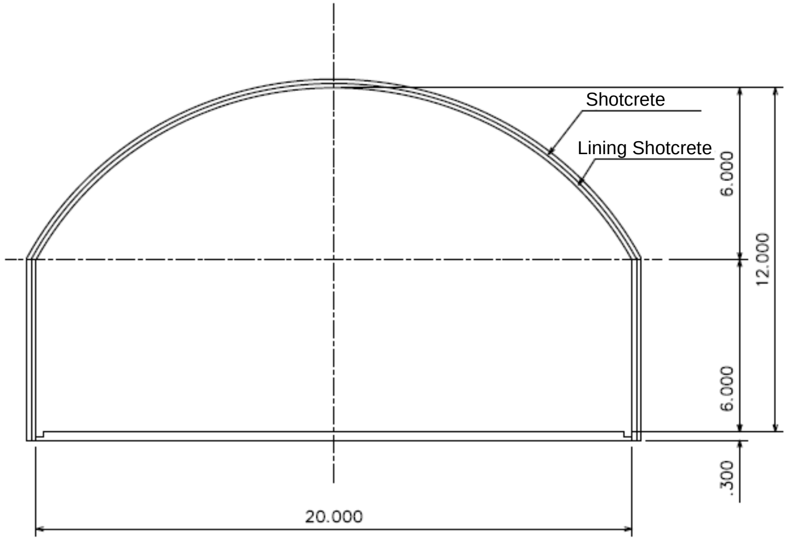
Table of the quantity of rock bolt materials

Name	Dimension	Standard	Unit	Quantity	Remarks
Rock bolt	L=4,000	A yield strength of 17.65 kN or more	pieces	10	Mortar adheres to all surfaces
Washer	150x150x9	ss400	pieces	10	
Nut		M24	pieces	10	

Table of the quantity of an AGF material

Improved length	Circumferential direction	Length direction	Diameter of a steel pipe	Thickness of a steel pipe		Unit	Quantity	Remarks
12.5m	@60cm	@6.0m	φ114.3	t=6.0		pieces	19	Impregnation method

Figure 7.8. Standard cross-sectional view (left) and support pattern (right) of the common section of an access tunnel connecting the detector hall and damping rings in the DIII section.



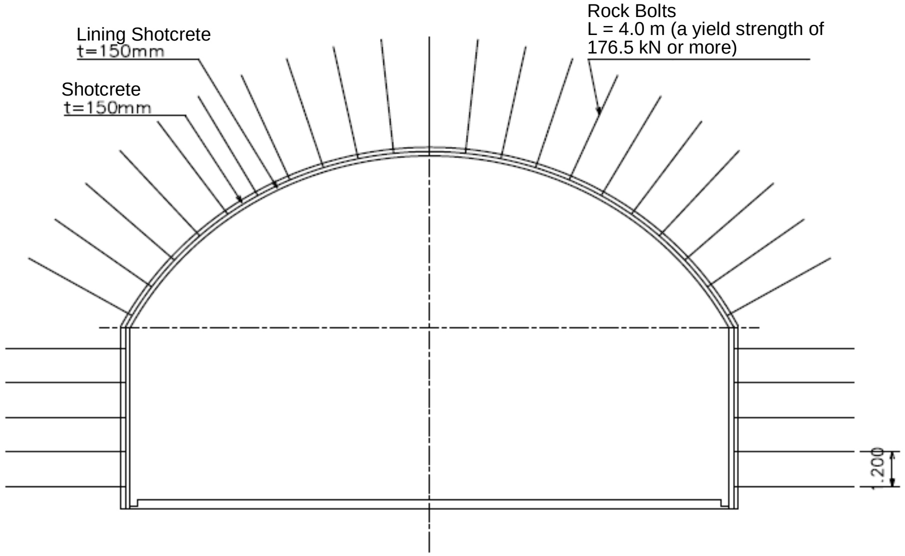


Table of unit quantities

Name	Specification	Unit	Quantity	Remarks
Cross section of excavation (design)		m^2	222.766	Per 1.0 m
Cross section of excavation (pay line)	Outbreak	m^2	231.213	Per 1.0 m
Steel arch support	H-125	units	-	Per 1.5 m
Shotcrete	t=150	m^2	37.573	Per 1.0 m
Rock bolt	L = 4.0 m, a yield strength of 17.65 kN or more	pieces		Per 1.5 m
Shot lining	t=150	m^3	37.573	Per 1.0 m
Lining concrete	t=300	m^3	-	Per 1.0 m
Roadbed concrete	t=400	m^3	7.9	Per 1.0 m

Table of specifications

Rock bolt			Steel arch support		Shot thickness	Wire net	Shot lining	Lining thickness		Assistance method
Length	Circumferential direction	Length direction	Upper half	Lower half				Arch/side wall	Invert	
4.0	1.5	1.5	-	-	15	-	15	-	-	-

Table of the amount of a shot material

Name	Dimension	Standard	Unit	Quantity	Remarks
Shotcrete	t=150	σ28=18N/mm2	m2/m	37.573	

Table of the quantity of rock bolt materials

Name	Dimension	Standard	Unit	Quantity	Remarks
Rock bolt	L=4,000	A yield strength of 17.65 kN or more	pieces	31	Mortar adheres to all surfaces
Washer	150x150x9	ss400	pieces	31	
Nut		M24	pieces	31	

Figure 7.9. Standard cross-sectional view (left) and support pattern (right) of an access hall (He dome).

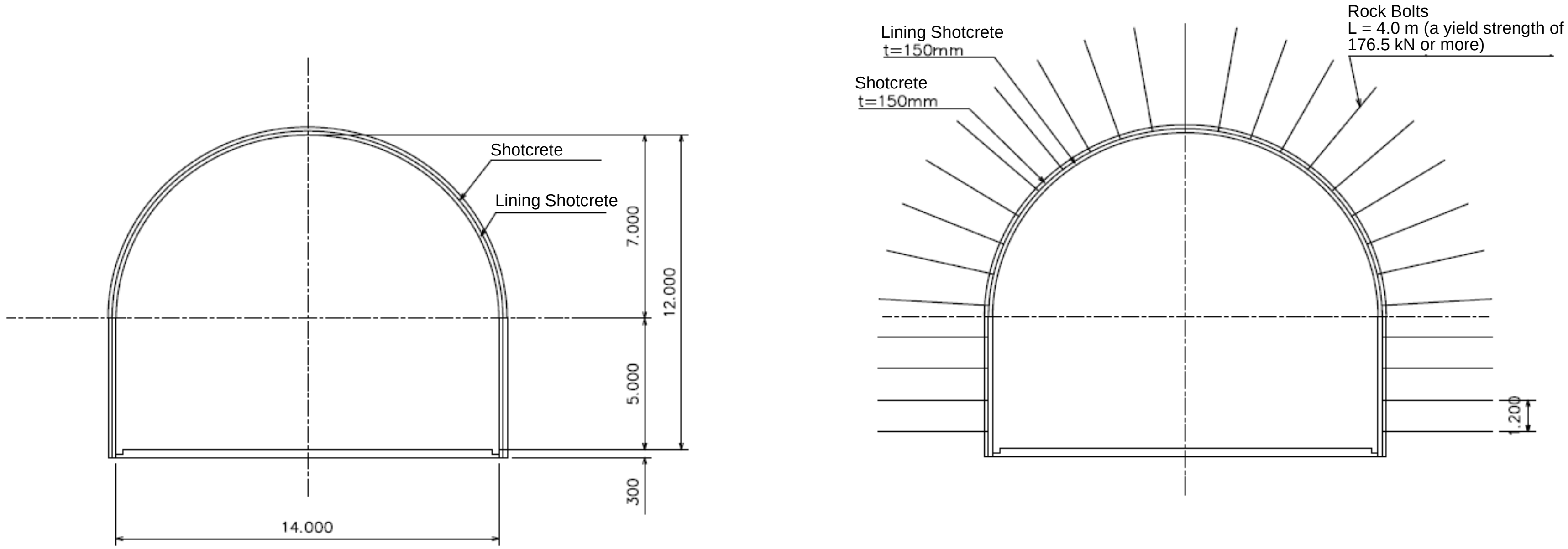


Table of unit quantities

Name	Specification	Unit	Quantity	Remarks
Cross section of excavation (design)		m^2	161.088	Per 1.0 m
Cross section of excavation (pay line)	Outbreak	m^2	168.541	Per 1.0 m
Steel arch support	H-125	units	-	Per 1.5 m
Shotcrete	t=150	m^2	33.062	Per 1.0 m
Rock bolt	L = 4.0 m, a yield strength of 17.65 kN or more	pieces	27	Per 1.5 m
Shot lining	t=100	m^3	33.062	Per 1.0 m
Lining concrete	t=300	m^3	-	Per 1.0 m
Roadbed concrete	t=400	m^3	5.5	Per 1.0 m

Table of specifications

Rock bolt			Steel arch support		Shot thickness	Wire net	Shot lining	Lining thickness		Assistance method
Length	Circumferential direction	Length direction	Upper half	Lower half				Arch/side wall	Invert	
4.0	1.5	1.5	-	-	15	-	15	-	-	-

Table of the amount of a shot material

Name	Dimension	Standard	Unit	Quantity	Remarks
Shotcrete	t=150	σ28=18N/mm2	m2/m	33.062	

Table of the quantity of rock bolt materials

Name	Dimension	Standard	Unit	Quantity	Remarks
Rock bolt	L=4,000	A yield strength of 17.65 kN or more	pieces	27	Mortar adheres to all surfaces
Washer	150x150x9	ss400	pieces	27	
Nut		M24	pieces	27	

Figure 7.10. Standard cross-sectional view (left) and support pattern (right) of an access hall (S/E/M dome).

7.2 Construction plan

7.2.1 Outline of the plan

The construction plan was created in accordance with the Civil Engineering Estimation Standards of the Ministry of Land, Infrastructure, Transport, and Tourism. For the construction of an access hall for an He dome, however, its support pattern, construction quantity per unit, and other matters were planned by setting the cycle time of construction in accordance with the above estimation standards because the construction requires a large cross section (222 m^2) of excavation.

The outline of the plan is as follows:

- Concepts of pay lines (for outbreak, extra shot, and extra lining) are in accordance with the above estimation standards.
- Standards on excavating equipment (drill carriages, wheel loaders, concrete guns, concrete breakers, and dump trucks) are in accordance with the above estimation standards.
- Standards on shotcrete, rock bolts, steel support, and other materials are in accordance with the above estimation standards.
- Excavation muck is assumed to be placed near a pithead temporarily, and then transported secondarily to a place 10 km away from the pithead by 10-ton dump trucks for disposal. Since the appearance of mineralized muck is assumed to be less likely according to the results of geological surveys, we do not consider the costs and facilities for mineralized muck disposal. If mineralized muck appears during construction, it is disposed of appropriately.

7.2.2 Preparation of pithead yards

For the preparation of pithead yards, we calculated the amount of cutting (88,000 m^3) and banking (2,200 m^3) for AS-12 as a typical place with average topography (ground surface inclination)[6)], and assumed the same amount of cutting and banking for yards of other points. AS-DR and AS-DH were determined to contain two temporary facilities in the same yard.

7.2.3 Excavation method

A blasting method is applied to excavate places other than pithead sections. For blasting, we have decided to apply a full face method with auxiliary bench cut for pattern C and a top heading and bench cut method for pattern D.

For the excavation of an access hall for an He dome, a top heading raising-cut and two-layered bench method was assumed.

[6] AS-12 is a pithead yard that was planned for a length of 30 km. Though the present project does not include this pithead, it does have average topography.

7.2.4 Temporary facility plan

Outline of the temporary facility plan is as follows:

- Types of temporary facilities on the pithead yards are in accordance with the above estimation standards.
- Temporary facilities in the shafts are in accordance with the above estimation standards. However, some parts of the specifications, such as electric equipment, were planned.
- Turbid water treatment facilities were planned by assuming the amount of treatment.

7.2.5 Process plan

Processes for each access tunnel were calculated for each cross section corresponding to a rock mass class on the basis of construction quantity per unit stated in the above estimation standards.

Access halls and the accelerator tunnel were assumed to be excavated at the same time after an access tunnel reaches the designated depth of the accelerator tunnel. Each pithead yard can ensure an area necessary for operation for the relevant facings. Excavation and lining shot are conducted before drainage (excavating the central drainage, installing perforated pipes, and backfilling broken stones) is performed; the floor slab concrete is then poured.

For the construction period for a large cross section of excavation for an access hall for an He dome, there are no methods of calculating its construction quantity per unit according to cycle time calculation at present. Accordingly, we calculated its cycle time and then the processes by applying the Civil Engineering Estimation Standards of the Ministry of Land, Infrastructure, Transport, and Tourism in the 2005 fiscal year, which was the last fiscal year when the calculation method of construction quantity per unit according to cycle time calculation was applicable.

In addition, the present construction process plan does not include the following activities: the new installation of roads from ordinary roads to pithead yards for construction; the widening and reinforcement of existing roads; and the rebuilding and new installation of bridges. This is because these activities are assumed to be conducted in advance by other construction work as necessary. Similarly, the present construction process plan does not include the periods necessary for obtaining lands for pithead yards, roads for construction, surplus soil disposal facilities, and other purposes.

Figure 7.11 shows a table of processes for the access tunnels and access halls. Construction periods depend on the lengths of the access tunnels; the shortest period was 33 months (AS+8) and the longest one was 54 months (AS-10).

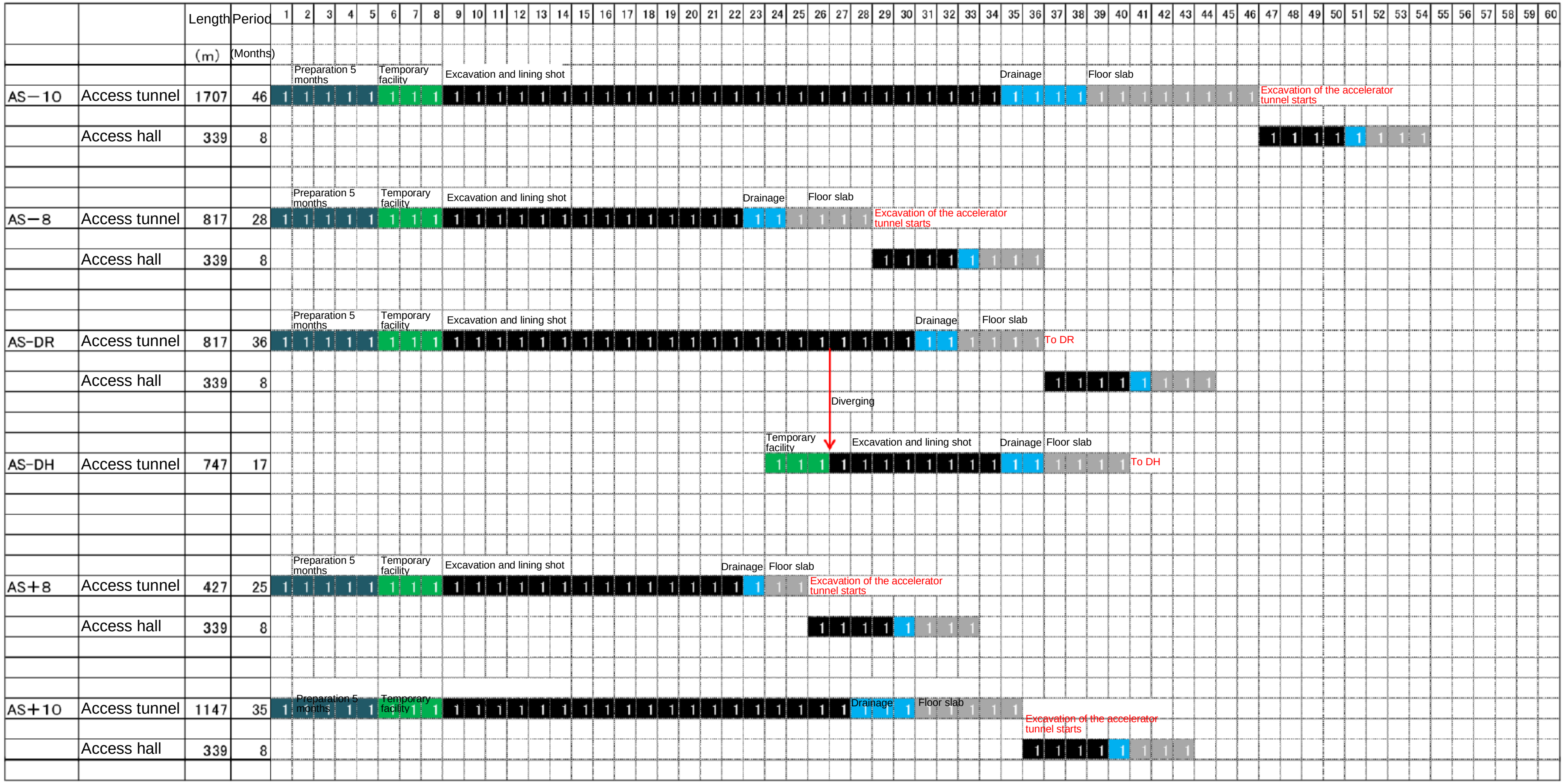


Figure 7.11. Processes for the access tunnels and access halls.

8 Accelerator Tunnel

8.1 Design specifications

8.1.1 Rock mass classes

The seismic exploration shows that an elastic wave speed at the altitude of the accelerator tunnel was 4.5 km/s or more in places other than the cross points of rivers, and the slowest speed was 2 km/s measured at the cross point of Satetsu River. Rock mass classes were determined to be CI for the whole site with reference to the Table of Rock Mass Class in the Civil Engineering Estimation Standards of the Ministry of Land, Infrastructure, Transport, and Tourism.

8.1.2 Dimensions and structures of the accelerator tunnel

The accelerator tunnel has the following four types of cross sections:

1. Main linear accelerator tunnels: a width of 9.5 m and a height of 5.5 m
2. BDS beam tunnels: a width of 8.0 m and a height of 5.0 m
3. BDS service tunnels and loop sections at both ends: a width of 4.5 m and a height of 4.0 m
4. Damping ring tunnel: a width of 5.5 m and a height of 4.7 m

In addition, the following dimensions are determined to be included:

- 10 widening sections are included for all accelerator-related tunnels; each section has a width of 11 m, a height of 5.5 m, and a length of 50 m.
- For the main linear accelerator tunnels, 76 reversal pits are included (at 200-m intervals in the overall length of 15,287 m); each of the pits has the same cross section as the tunnel (a width of 9.5 m and a height of 5.5 m) and a length of 20 m.

Table 8.1 shows the total lengths of the tunnels.

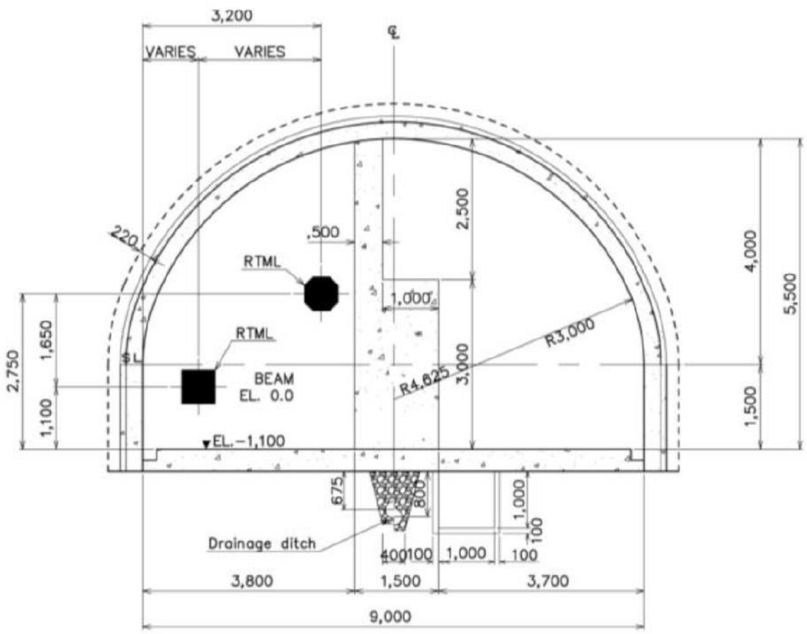
Table 8.1. List of the accelerator-related tunnels

Excavation section	Width (m)	Height (m)	Total length (m)
Main linear accelerator tunnels	9.5	5.5	15,287
BDS beam tunnels	8.0	5.0	5,850
BDS service tunnels	4.5	4.0	5,850
Damping ring tunnel	5.5	4.7	3,725
Loop sections at both ends	4.5	4.0	346
Widening sections	11.0	5.5	500
Reversal pits	9.5	5.5	1,520

We determined standard cross-sectional views and support patterns corresponding to rock mass class and excavation cross sections in accordance with the Civil Engineering Estimation Standards of the Ministry of Land, Infrastructure, Transport, and Tourism. Figures 8.1 to 8.4 show the standard cross-sectional views and support patterns of the tunnels[7)].

For cross points of rivers with a thin overburden, we additionally consider that they require other detailed geological surveys and design examination.

[7] As described in Chapter 1, the main linear accelerator tunnels require a width of 9.5 m. As described in Chapter 5, the result of the examination for a width of 9 m (Figure 8.1) was converted on the basis of a cross section for the evaluation of costs and excavation processes (per month); for the evaluation of roadbed processes, it was converted on the basis of a tunnel width.



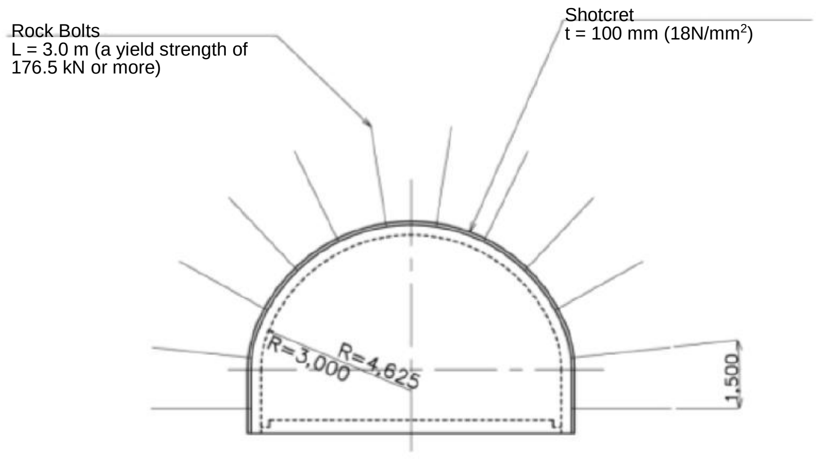


Table of unit quantities:

Name	Specification	Unit	Quantity	Remarks
Cross section of excavation (design)		m^2	51.941	Per 1.00 m
Cross section of excavation (pay line)	An outbreak of 22 cm	m^2	56.044	Per 1.00 m
Shotcrete	t = 100 mm	m^2	26.984	Per 1.50 m
Rock bolt	L = 3.0 m, a yield strength of 176.5 kN or more	pieces	12	Per 1.50 m
Lining concrete	t = 300 mm	m^2	5.255	Per 1.00 m
Roadbed concrete	t = 400 mm	m^2	3.500	Per 1.00 m
Central drainage (drain pipe)	High-density polyethylene pipe φ300 (perforated pipe)	m	10.0	Per 10.0 m
Central drainage (excavation)		m^3	6.956	Per 10.0 m
Central drainage (filter)	S-20	m^3	6.016	Per 10.0 m

Table of specifications:

Rock bolt			Steel support		Shot thickness (cm)	Lining thickness (cm)			Deformation margin (cm)		
Length	Circumferential direction	Length direction	Upper half	Lower half		Arch	Invert		Upper half	Lower half	Invert
3.0	1.5	1.5	-	-	10	30	-	-	0	0	-

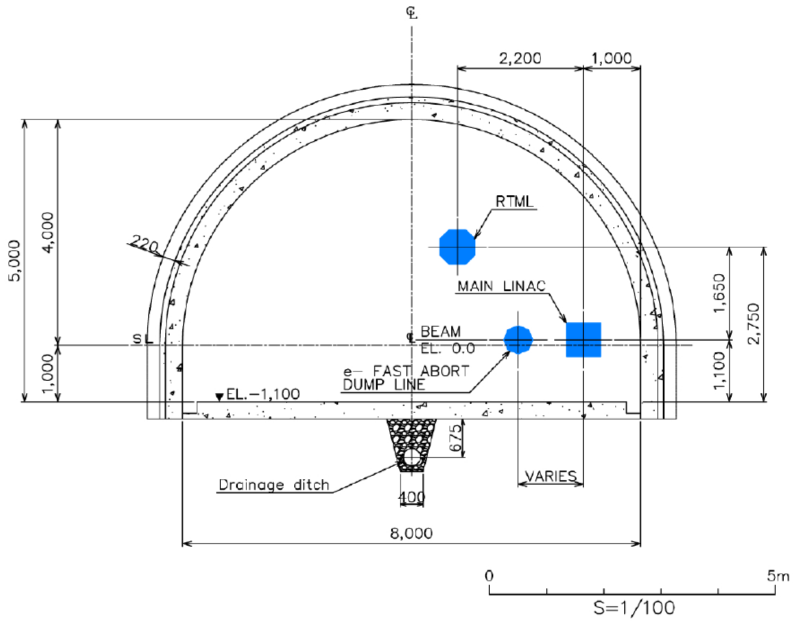
Table of shot material: Per P = 1.50 m

Name	Dimension	Standard	Unit	Quantity	Remarks
Shotcrete	t＝100	σ_{28}=18N/mm²	m^2	26.684	

Table of rock bolt materials: Per P = 1.50 m

Name	Dimension	Standard	Unit	Quantity	Remarks
Rock bolt	L=3,000	A yield strength of 176.5 kN or more	pieces	12	Mortar adheres to all surfaces
Washer	150x150x9	SS400	pieces	12	
Nut		M24	pieces	12	

Figure 8.1. Standard cross-sectional view (left) and support pattern (right) of a main linear accelerator tunnel and a reversal pit (as described in Chapter 5, the result of the examination for a width of 9 m was converted on the basis of the cross section and tunnel width).



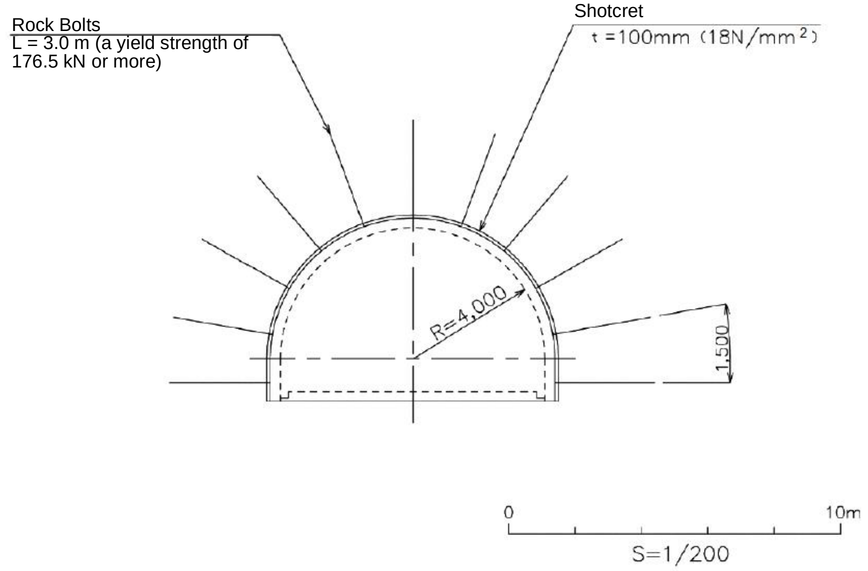


Table of specifications:

Rock bolt			Steel support		Shot thickness (cm)	Lining thickness (cm)			Deformation margin (cm)		
Length	Circumferential direction	Length direction	Upper half	Lower half		Arch	Invert		Upper half	Lower half	Invert
3.0	1.5	1.5	-	-	10	30	-	-	0	0	-

Table of unit quantities:

Name	Specification	Unit	Quantity	Remarks
Cross section of excavation (design)		m^2	41.851	Per 1.00 m
Cross section of excavation (pay line)	An outbreak of 22 cm	m^2	45.540	Per 1.00 m
Shotcrete	t = 100 mm	m^2	24.163	Per 1.50 m
Rock bolt	L = 3.0 m, a yield strength of 176.5 kN or more	pieces	11	Per 1.50 m
Lining concrete	t = 300 mm	m^2	4.691	Per 1.00 m
Roadbed concrete	t = 300 mm	m^2	2.300	Per 1.00 m
Central drainage (drain pipe)	High-density polyethylene pipe φ300 (perforated pipe)	m	10.0	Per 10.0 m
Central drainage (excavation)		m^3	5.839	Per 10.0 m
Central drainage (filter)	S-20	m^3	4.899	Per 10.0 m

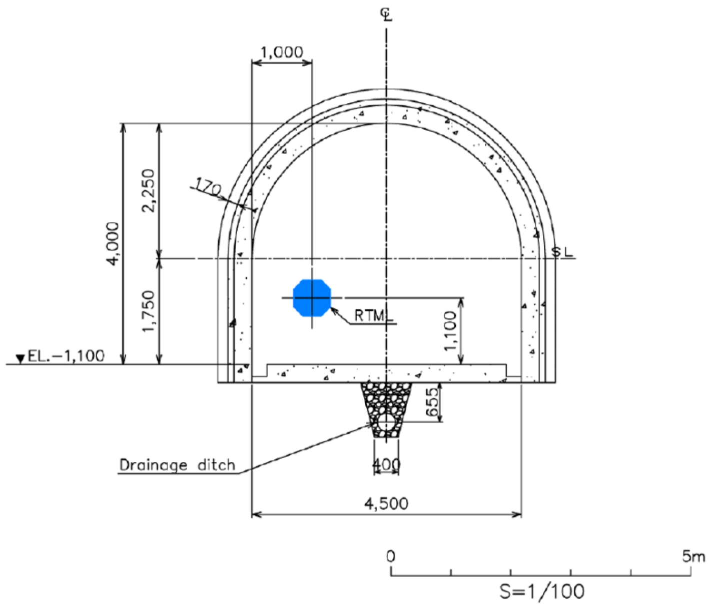
Table of shot material: Per P = 1.50 m

Name	Dimension	Standard	Unit	Quantity	Remarks
Shotcrete	t = 100	σ_{28}=18N/mm^2	m^2	24.163	

Table of rock bolt materials: Per P = 1.50 m

Name	Dimension	Standard	Unit	Quantity	Remarks
Rock bolt	L=3,000	A yield strength of 176.5 kN or more	pieces	11	Mortar adheres to all surfaces
Washer	150x150x9	SS400	pieces	11	
Nut		M24	pieces	11	

Figure 8.2. Standard cross-sectional view (left) and support pattern (right) of a BDS beam tunnel.



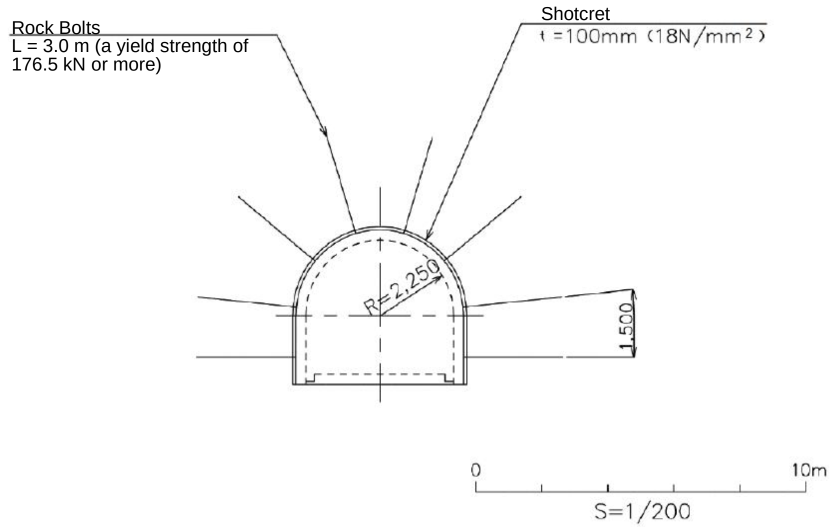


Table of specifications:

Rock bolt			Steel support		Shot thickness (cm)	Lining thickness (cm)			Deformation margin (cm)		
Length	Circumferential direction	Length direction	Upper half	Lower half		Arch	Invert		Upper half	Lower half	Invert
3.0	1.5	1.5	-	-	10	30	-	-	0	0	-

Table of unit quantities:

Name	Specification	Unit	Quantity	Remarks
Cross section of excavation (design)		m^2	21.896	Per 1.00 m
Cross section of excavation (pay line)	An outbreak of 17 cm	m^2	24.054	Per 1.00 m
Shotcrete	t = 100 mm	m^2	18.167	Per 1.50 m
Rock bolt	L = 3.0 m, a yield strength of 176.5 kN or more	pieces	8	Per 1.50 m
Lining concrete	t = 300 mm	m^2	3.492	Per 1.00 m
Roadbed concrete	t = 300 mm	m^2	1.250	Per 1.00 m
Central drainage (drain pipe)	High-density polyethylene pipe φ300 (perforated pipe)	m	10.0	Per 10.0 m
Central drainage (excavation)		m^3	5.668	Per 10.0 m
Central drainage (filter)	S-20	m^3	4.727	Per 10.0 m

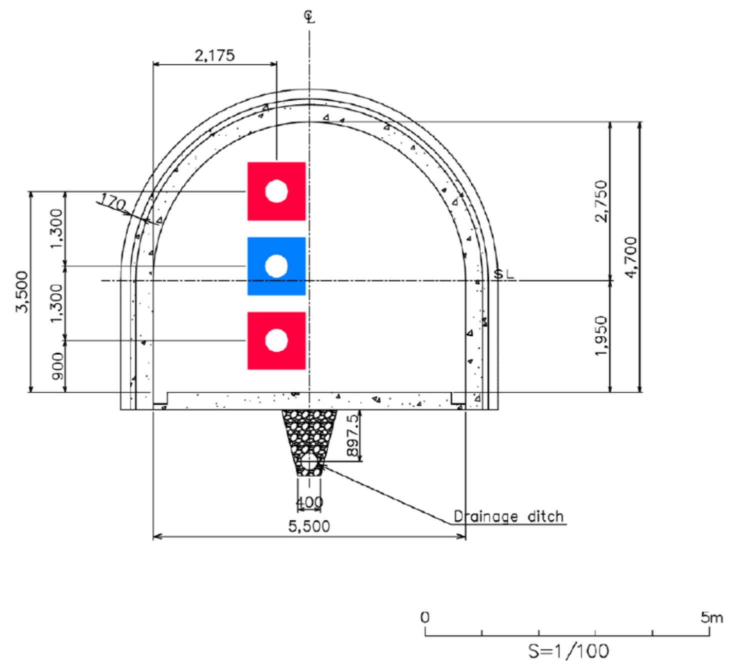
Table of shot material: Per P = 1.50 m

Name	Dimension	Standard	Unit	Quantity	Remarks
Shotcrete	t＝100	σ_{28}=18N/mm^2	m^2	18.167	

Table of rock bolt materials: Per P = 1.50 m

Name	Dimension	Standard	Unit	Quantity	Remarks
Rock bolt	L=3,000	A yield strength of 176.5 kN or more	pieces	8	Mortar adheres to all surfaces
Washer	150x150x9	SS400	pieces	8	
Nut		M24	pieces	8	

Figure 8.3. Standard cross-sectional view (left) and support pattern (right) of a BDS service tunnel and a loop section at both ends.



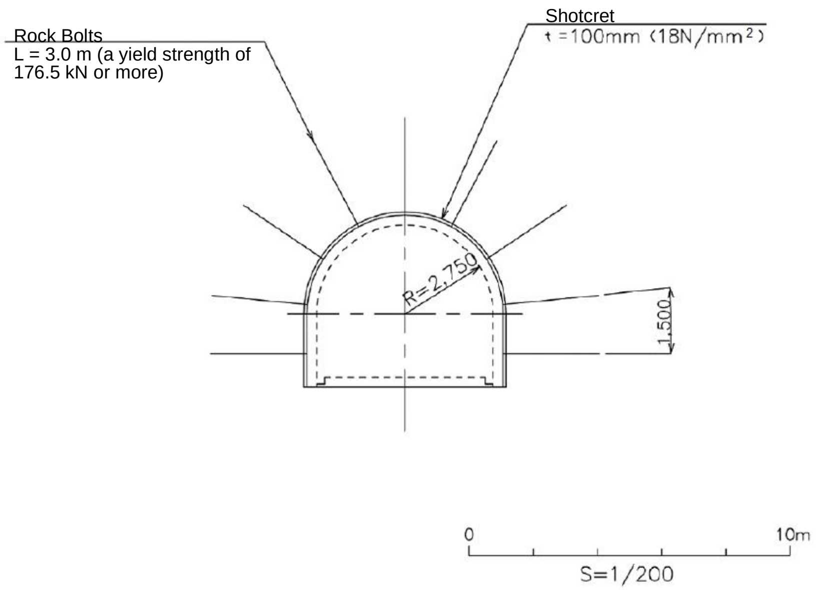


Table of unit quantities:

Name	Specification	Unit	Quantity	Remarks
Cross section of excavation (design)		m^2	29.761	Per 1.00 m
Cross section of excavation (pay line)	An outbreak of 17 cm	m^2	32.254	Per 1.00 m
Shotcrete	t = 100 mm	m^2	21.123	Per 1.50 m
Rock bolt	L = 3.0 m, a yield strength of 176.5 kN or more	pieces	9	Per 1.50 m
Lining concrete	t = 300 mm	m^2	4.083	Per 1.00 m
Roadbed concrete	t = 300 mm	m^2	1.550	Per 1.00 m
Central drainage (drain pipe)	High-density polyethylene pipe φ300 (perforated pipe)	m	10.0	Per 10.0 m
Central drainage (excavation)		m^3	7.882	Per 10.0 m
Central drainage (filter)	S-20	m^3	6.942	Per 10.0 m

Table of specifications:

Rock bolt			Steel support		Shot thickness (cm)	Lining thickness (cm)			Deformation margin (cm)		
Length	Circumferential direction	Length direction	Upper half	Lower half		Arch	Invert		Upper half	Lower half	Invert
3.0	1.5	1.5	-	-	10	30	-	-	0	0	-

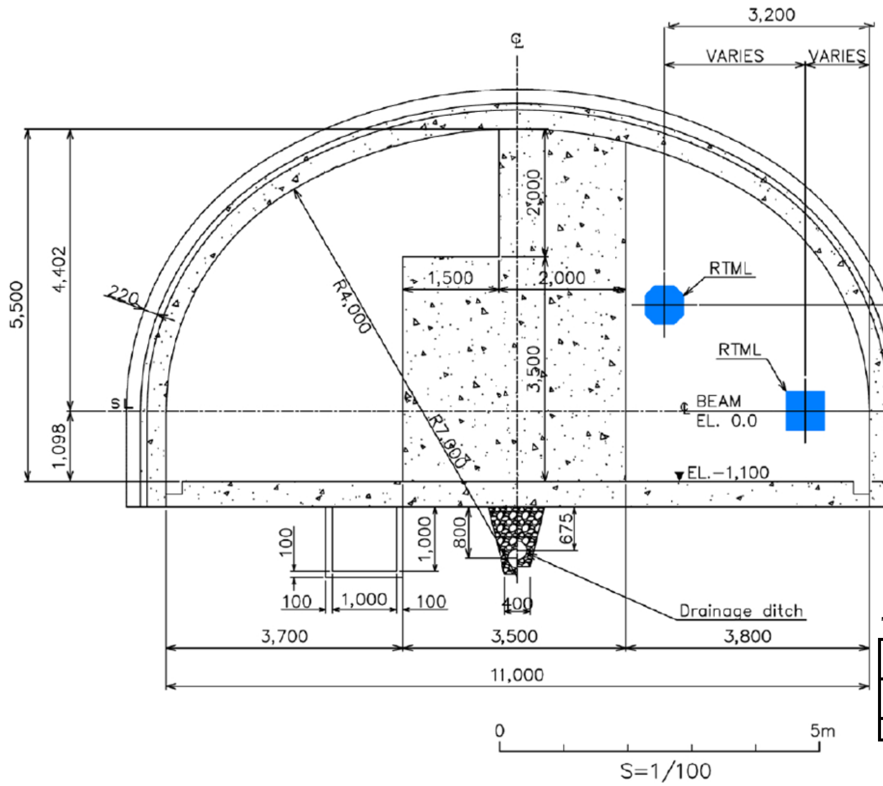
Table of shot material: Per P = 1.50 m

Name	Dimension	Standard	Unit	Quantity	Remarks
Shotcrete	t = 100	σ_{28}=18N/mm^2	m^2	21.123	

Table of rock bolt materials: Per P = 1.50 m

Name	Dimension	Standard	Unit	Quantity	Remarks
Rock bolt	L=3,000	A yield strength of 176.5 kN or more	pieces	9	Mortar adheres to all surfaces
Washer	150x150x9	SS400	pieces	9	
Nut		M24	pieces	9	

Figure 8.4. Standard cross-sectional view (left) and support pattern (right) of the damping ring tunnel.



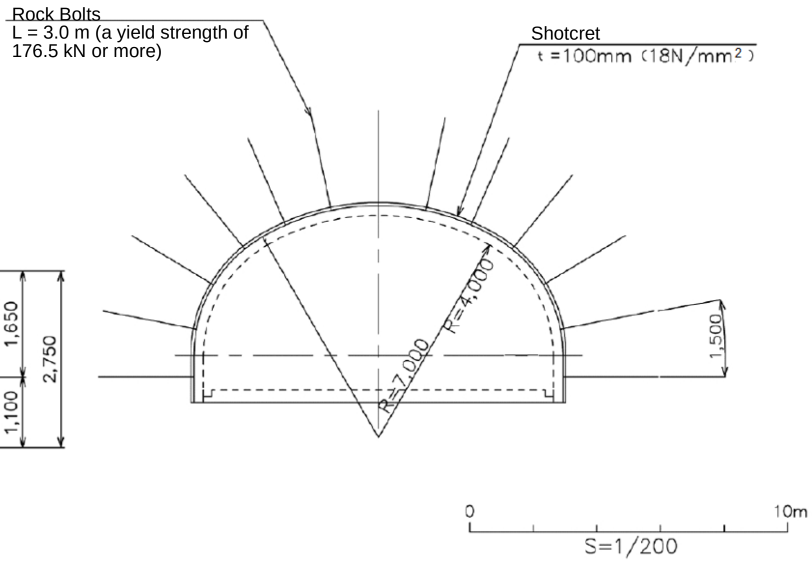


Table of specifications:

Rock bolt			Steel support		Shot thickness (cm)	Lining thickness (cm)			Deformation margin (cm)		
Length	Circumferential direction	Length direction	Upper half	Lower half		Arch	Invert		Upper half	Lower half	Invert
3.0	1.5	1.5	-	-	10	30	-	-	0	0	-

Table of unit quantities:

Name	Specification	Unit	Quantity	Remarks
Cross section of excavation (design)		m^2	62.725	Per 1.00 m
Cross section of excavation (pay line)	An outbreak of 22 cm	m^2	67.194	Per 1.00 m
Shotcrete	t = 100 mm	m^2	29.476	Per 1.50 m
Rock bolt	L = 3.0 m, a yield strength of 176.5 kN or more	pieces	13	Per 1.50 m
Lining concrete	t = 300 mm	m^2	5.753	Per 1.00 m
Roadbed concrete	t = 400 mm	m^2	4.300	Per 1.00 m
Central drainage (drain pipe)	High-density polyethylene pipe φ300 (perforated pipe)	m	10.0	Per 10.0 m
Central drainage (excavation)		m^3	6.956	Per 10.0 m
Central drainage (filter)	S-20	m^3	6.016	Per 10.0 m

Table of shot material: Per P = 1.50 m

Name	Dimension	Standard	Unit	Quantity	Remarks
Shotcrete	t＝100	σ_{28}=18N/mm^2	m^2	29.476	

Table of rock bolt materials: Per P = 1.50 m

Name	Dimension	Standard	Unit	Quantity	Remarks
Rock bolt	L=3,000	A yield strength of 176.5 kN or more	pieces	13	Mortar adheres to all surfaces
Washer	150x150x9	SS400	pieces	13	
Nut		M24	pieces	13	

Figure 8.5. Standard cross-sectional view (left) and support pattern (right) of a widening section.

8.2 Construction plan

8.2.1 Outline of the plan

The construction plan was created in accordance with the Civil Engineering Estimation Standards of the Ministry of Land, Infrastructure, Transport, and Tourism. Table 8.2 lists the construction specifications of the excavation sections. However, the above standards state only those for a one-way tunneling length of up to 2,500 m. For cases where the one-way tunneling length exceeds 2,500 m, we examined the plan by separately considering an increase in the number of dump trucks, tunnel special processes, and other matters.

Table 8.2. List of the construction specifications of the accelerator-related tunnels

Excavation section	Cross section (m^2)	Applicable cross section (m^2)	Excavation method	Mucking method	Excavation speed (m/month)
Main linear accelerator tunnels	55.75	65.0	NATM	Tire	106
BDS beam tunnels	41.85	40.0	NATM	Tire	124
BDS service tunnels	21.90	20.0	NATM	Rail	73
Damping ring tunnel	29.76	30.0	NATM	Rail	62
Loop sections at both ends	21.90	20.0	NATM	Rail	73

In addition, outlines of the plan are as follows:

- Temporary facility yards for tunnels are to be used continuously from the construction of the access tunnels. Additionally, we do not consider the restoration of the temporary facility yards.
- Excavation muck is assumed to be placed near a pithead temporarily, and then transported secondarily to a place 10 km away from the pithead by 10-ton dump trucks to be disposed of. Since the appearance of mineralized muck is assumed to be less possible according to the results of geological surveys, we do not consider costs and facilities for mineralized muck disposal. If mineralized muck appears during construction, it is disposed of appropriately.

8.2.2 Outline of the construction process plan

The construction process plan was according to the following conditions:

Excavation from AH+10/AH-10

1. A main linear accelerator tunnel is excavated in the two directions simultaneously to one end and the center.

2. After the excavation reaches the end, a loop section is excavated.
3. Lining will be completed three months after the completion of the excavation.
4. After the lining is completed, roadbed concrete is constructed at a speed of 21 m/day simultaneously in the two directions to the end and center.

Excavation from AH+8/AH-8

1. The following three routes of excavation are conducted simultaneously: a main linear accelerator tunnel to one end, and a BDS beam tunnel and a BDS service tunnel to the center.
2. Lining will be completed three months after the completion of excavation.
3. After the lining of these tunnels is completed, roadbed concrete is constructed at a speed of 21 m/day simultaneously in the two directions to the end and center.

Excavation of the damping ring tunnel

1. Four faces are excavated simultaneously.
2. The lining will be completed three months after the completion of the simultaneous excavation.
3. After the lining is completed, the roadbed concrete is constructed at a speed of 21 m/day simultaneously for the four places.

In addition, we considered applying both a tire method for the main linear accelerator tunnels and a rail method for the loop sections as excavation methods from AH+10/AH-10 to the ends. The details of these excavation methods are supposed to require re-examination after construction methods and processes are determined.

Figure 8.6 shows the accumulated construction period from the beginning of the construction of the access tunnels. Since the period for starting the construction of the accelerator tunnel depends on the construction period of the access tunnels, the construction period of the accelerator tunnel is decided by the lengths of the access tunnels. One access tunnel is the longest with a route where excavation starts from AS-10 to a RTML at one end; this route had the longest accumulated construction period of 90.8 months.

	(Unit)	−	PM−10	+	−	PM−8	+	−		DR		+	−	PM+8	+	−	PM+10	+
Length	(m)		1,707			817				817				427			1,147	
Construction period	(month)		46.0			28.0				36.0				25.0			35.0	
									Vertical section	533								
									Straight section	380	1/4	4 facings						
									Curved section	418	1/4	4 facings						
							3,489	76		1,331		76	2,361					
Section length	(m)			4,950			3,140	273	Parallel		Parallel	273	2,012		4,795			
Divided section length	(m)	2,438+RT173		1,519	3,431		3,140	294				294	2,012		2,929	1,867		2,516+RT173
Shape of excavation cross section																		
Semicylindrical shape (106.2 m/month)	(month)	23.0		14.3	32.3			3.0				3.0			27.6	17.6		23.7
DR (62 m/month)	(month)									21.5								
RTML (73 m/month)	(month)	2.4					43.0						27.6					2.4
Excavation period	(month)	71.4		60.3	60.3		71.0	39.0		57.5		39.0	52.6		52.6	52.6		61.1
Lining		3.0		3.0	3.0		3.0	3.0		3.0		3.0	3.0		3.0	3.0		3.0
(conducted 3 months after the excavation)	(month)																	
Roadbed construction		6.2		5.9	5.9		6.3	0.7		3.0		0.7	4.5		5.7	5.7		6.4
(ML 418 m/month, BDS 496 m/month)																		
Curtain wall construction	(month)	10.2		10.4	10.4			1.2				1.2			10.0	10.0		10.5
(PC block 239 m/month)																		
Overall period	(month)	90.8		79.6	79.6		77.3	43.9		63.5		43.9	60.1		71.3	71.3		81.0
	(year)	7.6		6.6	6.6		6.4	3.7		5.3		3.7	5.0		5.9	5.9		6.8

Figure 8.6.　Processes of the accelerator tunnel.

9 Detector Hall and Peripheral Tunnels

9.1 Design specifications

9.1.1 Rock mass classes

The results of the seismic exploration and boring surveys show that hard granite with a few fissures are widely distributed at the depth where the detector hall is constructed. We regarded the relevant rock masses as granite with classes B to CH with reference to the result of the geological surveys. In addition, there is an overburden of about 100 m.

For tunnels constructed around the detector hall, we regarded the rock mass class as CI, which is same as other accelerator-related tunnels.

9.1.2 Dimensions and structures of the detector hall and peripheral tunnels

The specifications of the detector hall and peripheral tunnels are as follows:

1. Detector hall:

   Space size: W25 m x L133 m x H42 m

   Shape: Bullet shape (with an arch and a vertical wall), which has a mushroom-shaped shaft through part (with ceiling concrete)

   - The ceiling is finished with shotcrete, and construction finishing work is conducted separately.
   - The building of a crane wall and bottom slab concrete is included.
2. Main shaft: a diameter of 18 m and a depth of approximately 70 m
3. Utility shaft: a diameter of 10 m and a depth of approximately 70 m
4. Peripheral tunnel: a width of 8.0 m and a height of 7.5 m
5. Peripheral tunnel: a width of 4.0 m and a height of 4.0 m

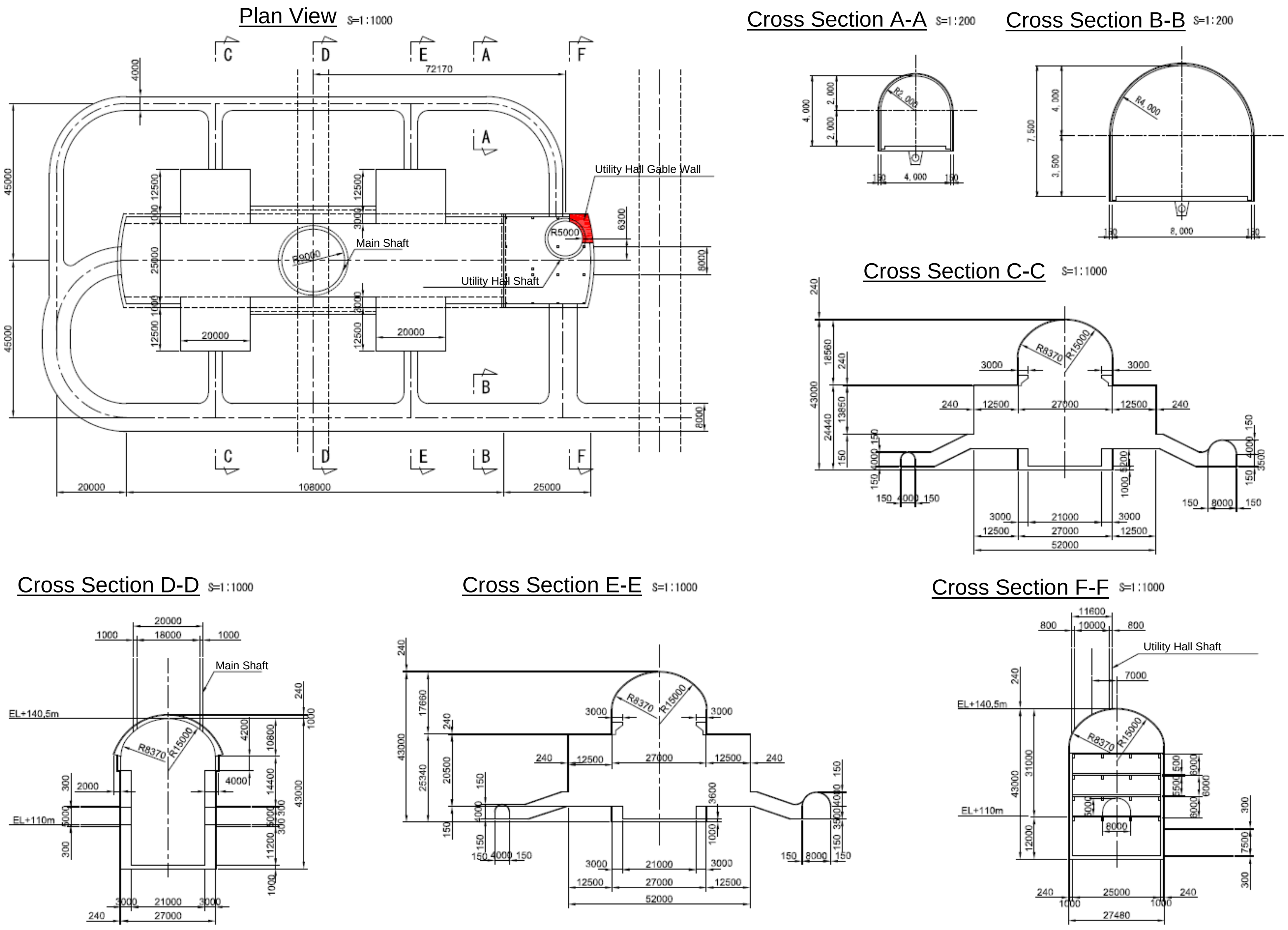


Figure 9.1. A schematic drawing of the detector hall and peripheral tunnels.

9.1.3 Design of support

Detector hall

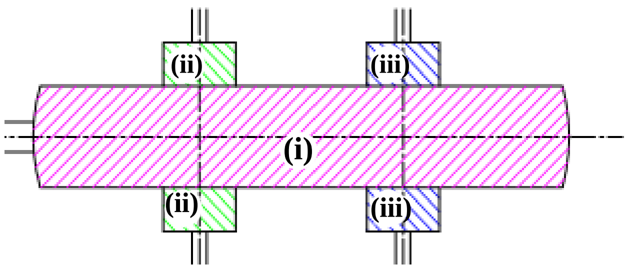
Rock bolts, PS anchors, and shotcrete were planned to be applied to the support for the detector hall, with reference to the construction case of the Okawachi Power Station (in the Kanzaki District of Hyogo Prefecture), which has a bullet shape the same as the detector hall and dimensions also similar to it.

In addition, we decided to support the loads of the main shaft and utility shaft connecting to the detector hall with shearing resistance between the rock mass and support around the shafts and the detector hall. To support shafts, the structure of the detector hall was planned as follows:

- The load applied from the main shaft to the detector hall is supported by lining concrete (with the crown 1 m thick) poured into the arch connecting to the shaft and the side wall (with a thickness of 3 m) of the detector hall.
- The load from the utility shaft to the detector hall is supported by the gable wall connecting to the shaft. Table 9.1 and Figures 9.2 to 9.4 show standard support patterns.

Table 9.1. List of standard support patterns of the detector hall.

Part	Rock bolt			PS anchor		Shotcrete σ_{ck} = 18 N/mm^2
	Length (m)	Yield strength (kN)	Circumferential/ length direction Interval (m)	Length (m)	Circumferential/ length direction Interval (m)	Thickness (cm)
(i)	5.0	120	2 / 2	15.0	4 / 4	24
Gable of (i)	5.0	120	2 / 2	15.0	4 / 4	24
(ii)	5.0	120	2 / 2	15.0	4 / 4	24
Gable of (ii)	5.0	120	2 / 2	15.0	4 / 4	24
(iii)	5.0	120	2 / 2	15.0	4 / 4	24
Gable of (iii)	5.0	120	2 / 2	15.0	4 / 4	24



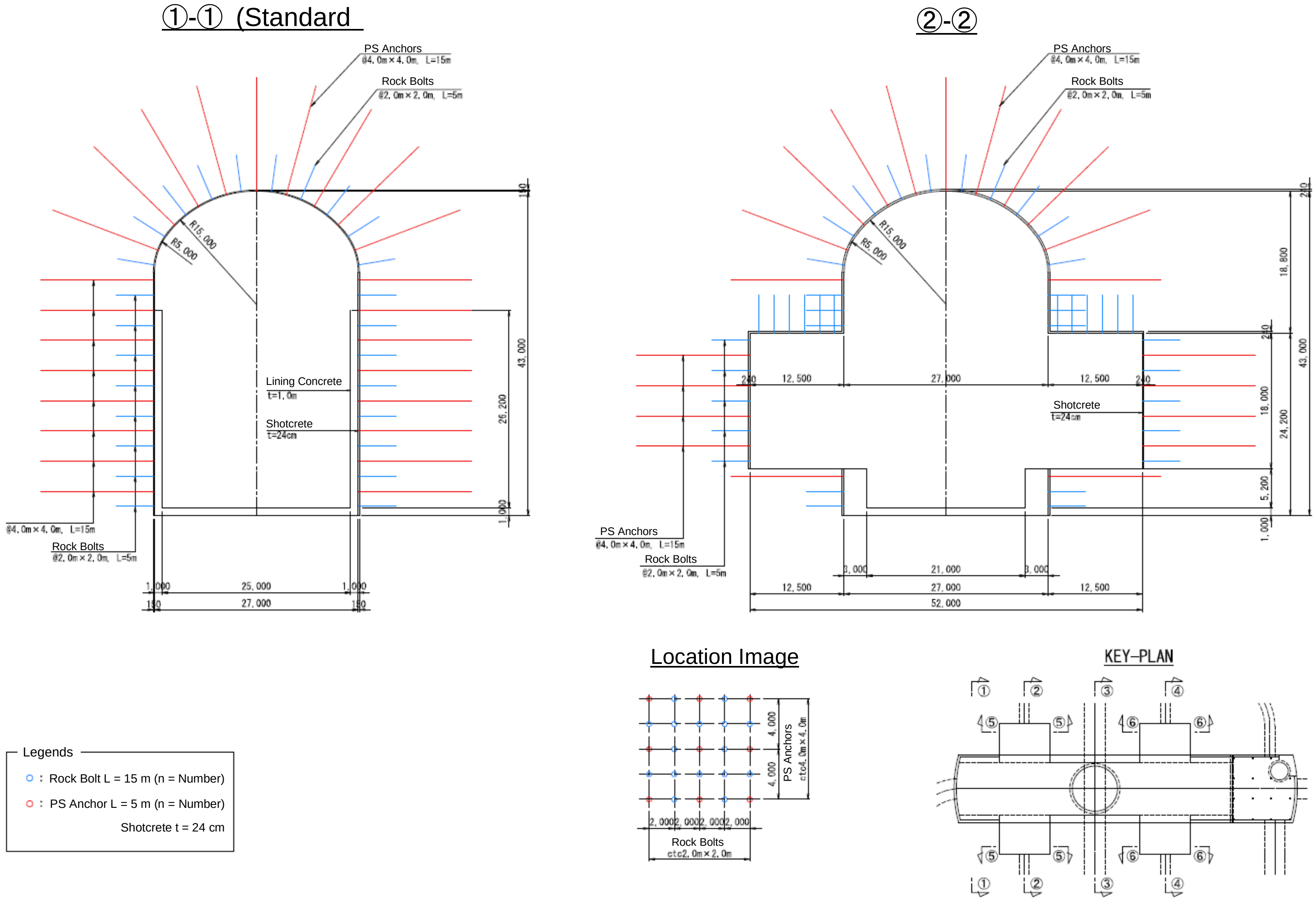


Figure 9.2. Standard support pattern (1) of the detector hall.

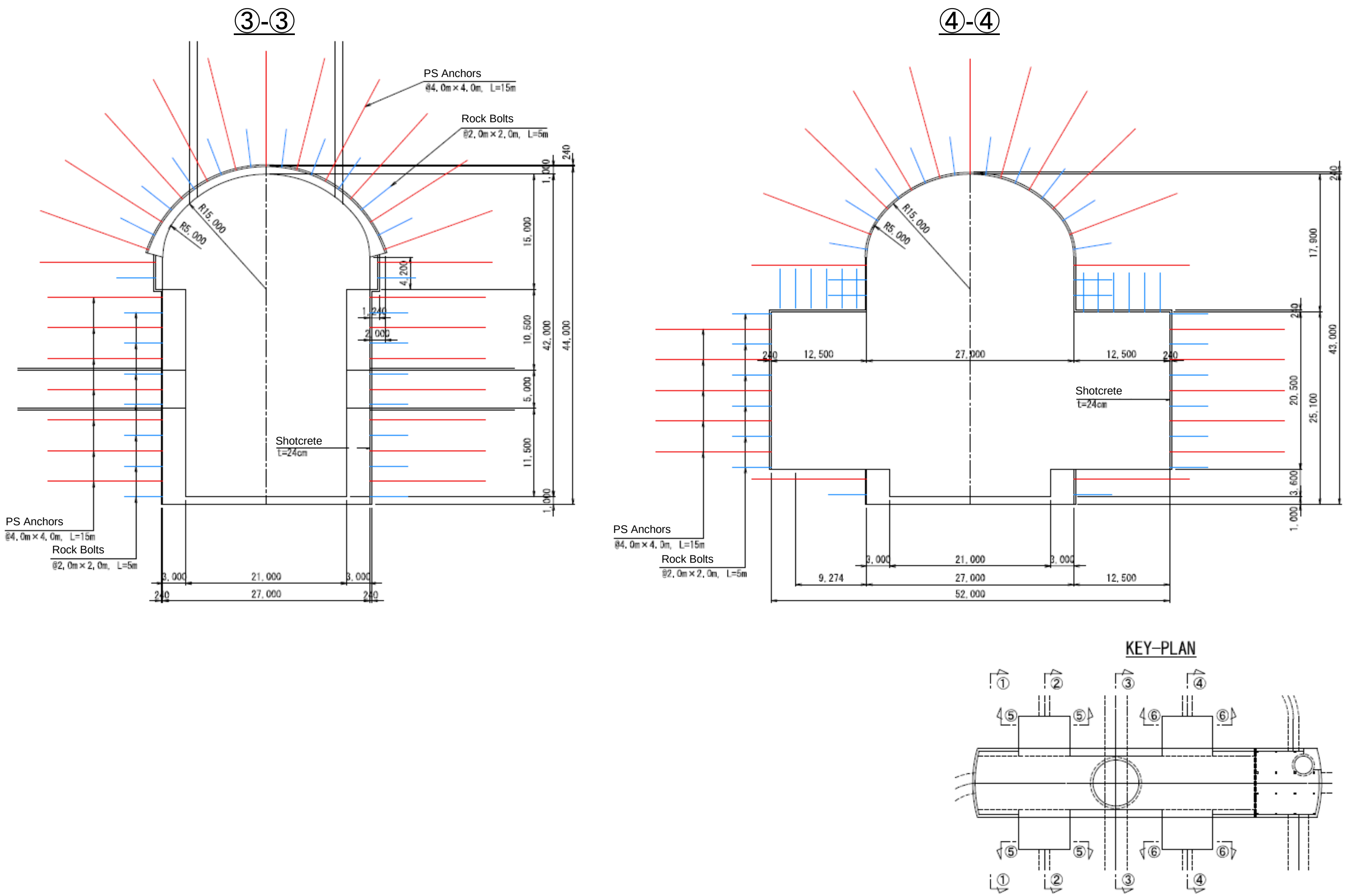


Figure 9.3.　Standard support pattern (2) of the detector hall.

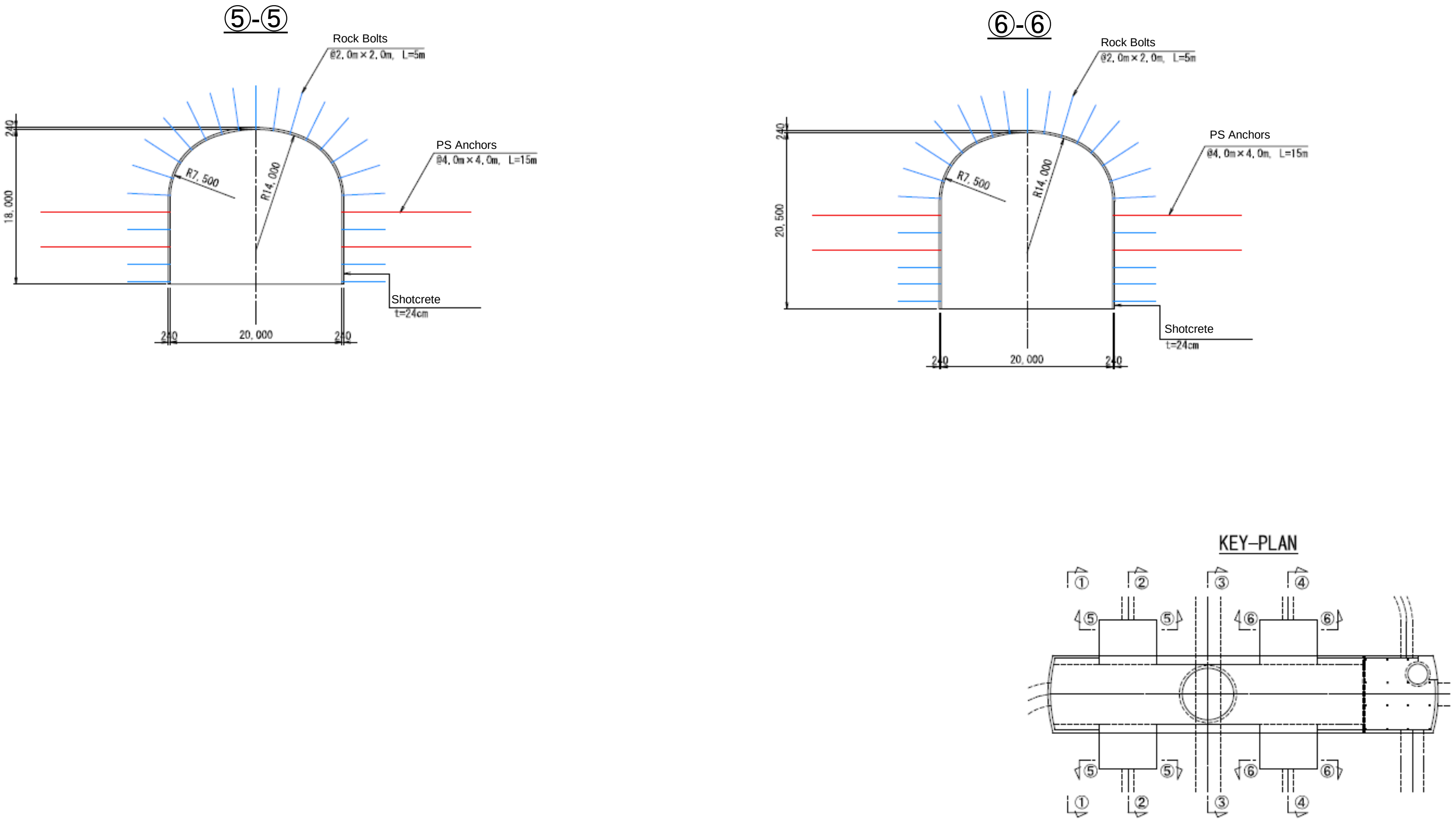


Figure 9.4. Standard support pattern (3) of the detector hall.

Shaft

Support structures of the shafts were examined by applying the Road Tunnel Technical Standards for similar cross-sectional dimensions because shafts are not covered by the Civil Engineering Estimation Standards of the Ministry of Land, Infrastructure, Transport, and Tourism.

- Since the main shaft has a large cross section with a diameter of 18 m, its support pattern was determined in accordance with the standard support patterns for large cross-sectional tunnels in the Design Guidelines of the East, Central, and West Nippon Expressway Company.
- The support pattern of the utility shaft (with a diameter of 10 m) was determined in accordance with the “Standard Support Patterns for Road Tunnels (with medium cross sections)” in the Road Tunnel Technical Standards of the Japan Road Association.

Figure 9.5 shows the tunnel support patterns of the main shaft and utility shaft.

For the cross areas of the crown of the detector hall and the shafts, in addition, we consider that they require other detailed examinations.

Peripheral tunnels

Standard support patterns of peripheral tunnels were in accordance with the Road Tunnel Technical Standards of the Japan Road Association, and determined to be the rock mass class of CI. Figure 9.6 shows the support patterns of peripheral tunnels.

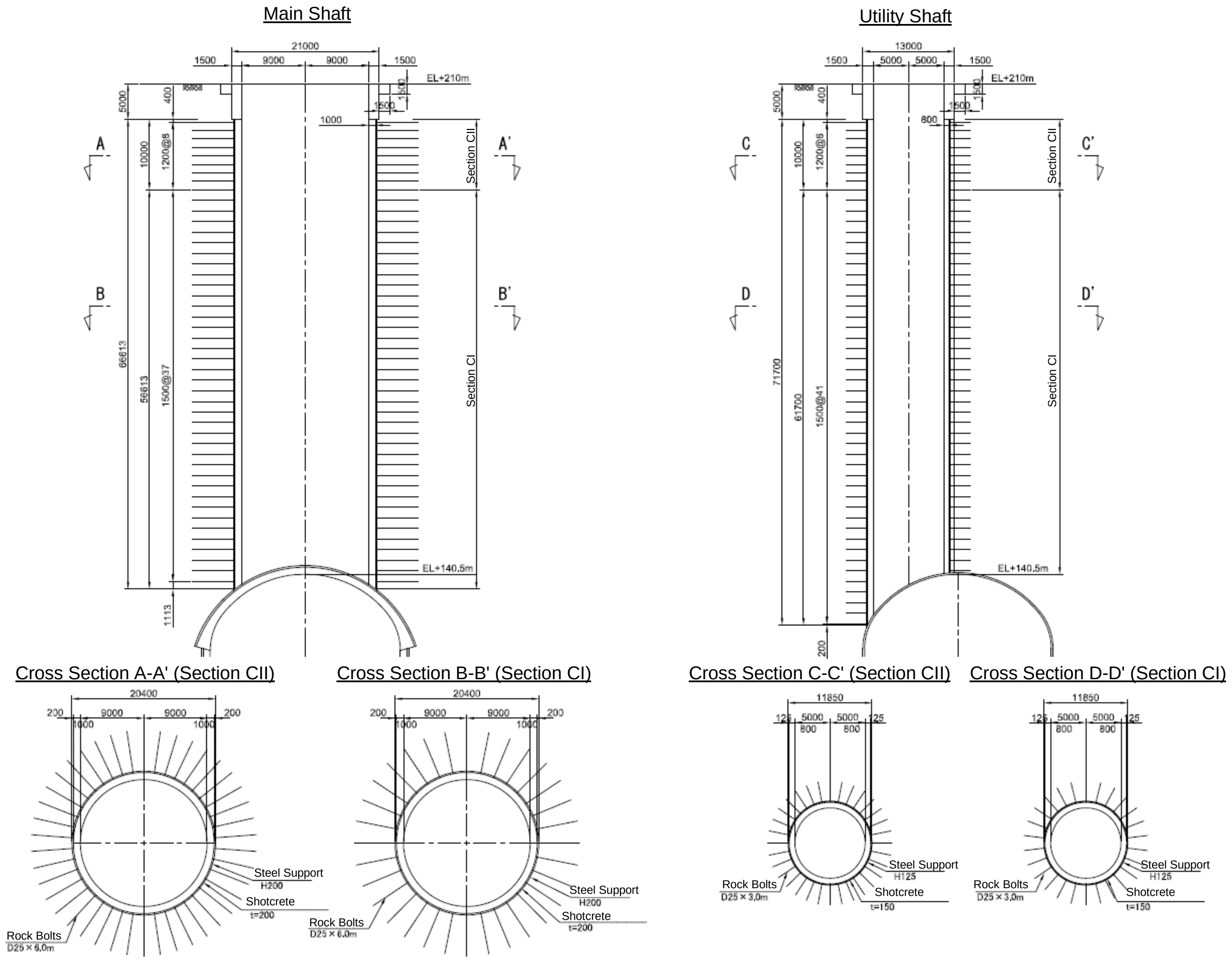


Figure 9.5. Standard support patterns of the main shaft and utility shaft.

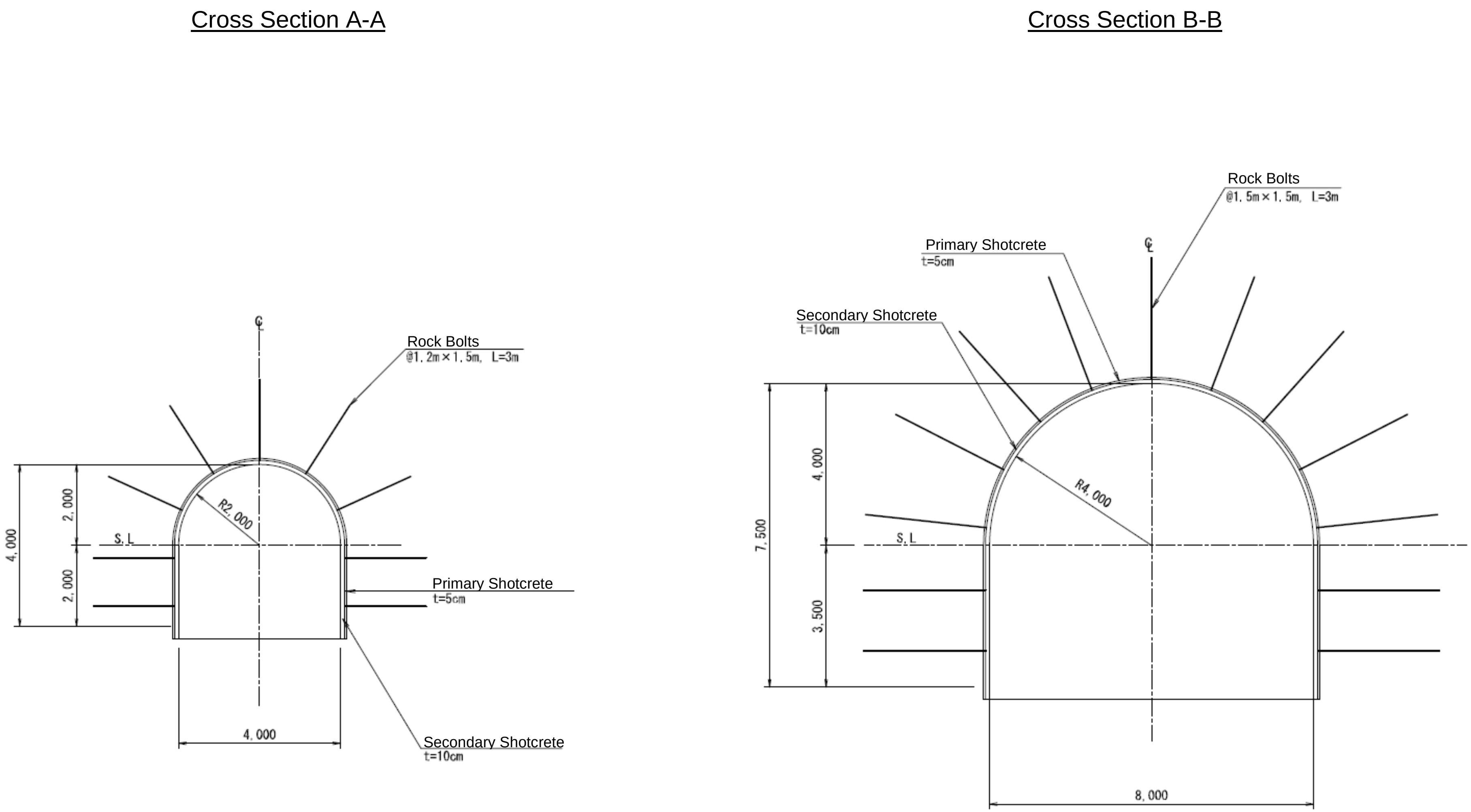


Figure 9.6. Standard support patterns of peripheral tunnels.

9.2 Construction plan

9.2.1 Outline of the construction

Applied construction methods were according to the following conditions:

- A full face method was applied for shaft excavation because no under-work pits are used for the excavation.
- The arch of the detector hall was constructed by a center drift advancing method after the shafts are excavated, and a basement for the excavation of the arch is then installed under the shafts. Since an access tunnel and peripheral tunnels reach the lower stage during the construction of the body, mucking holes are made and a bench excavation method is applied for the construction.
- Excavation muck from the arch is removed from the shafts, and that from the benches is removed from a lower access tunnel.
- Excavation muck is assumed to be placed near a pithead temporarily, and then transported secondarily to a place 10 km away from the pithead by 10-ton dump trucks for disposal. Since the appearance of mineralized muck is assumed to be less likely according to the results of geological surveys, we do not consider the costs and facilities for mineralized muck disposal. If mineralized muck appears during construction, it is disposed of appropriately.

9.2.2 Outline of the construction process plan

For the excavation speed of a shaft, we determined 18.9 m/month for an initial liner section and 11.6 m/month for an NATM section with reference to construction work for the tunnel discharging facility gate house and other sections for the redevelopment of the Amagase Dam (in the Uji City of Kyoto Prefecture) (a depth of 49.9 m, a finished inner diameter of 24 m, and an excavation outer diameter of 26.4 m).

Figure 9.7 shows a list of the processes for the detector hall, shafts, and peripheral tunnels. Construction period was estimated at 66 months.

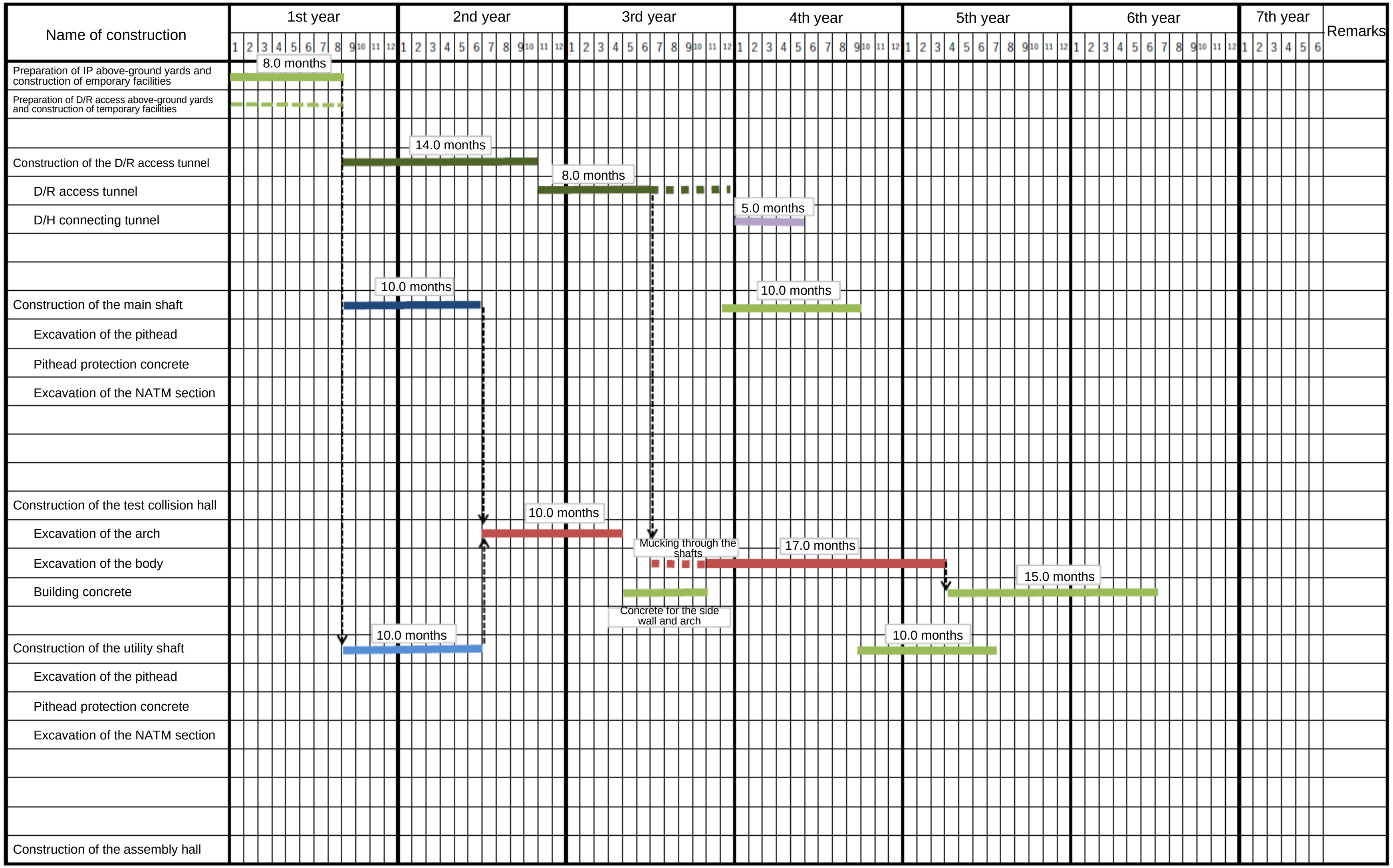


Figure 9.7. Processes for the detector hall, shafts, and peripheral tunnels

10 Drainage Facilities

10.1 Requirements of the ILC accelerator for drainage facilities

10.1.1 Accelerator tunnel

The main linear accelerator tunnels, which contain accelerator bodies (cryomodules) using liquid helium, are parallel to the geoid plane. In addition, the BDS tunnels (occupying approximately 6 km near the center) are determined to be “laser straight,” and have a falling gradient of approximately 1/2,000 towards the center. A drainage method is thus required to ensure the drainage of groundwater into the accelerator tunnel, which is horizontal or has a very low inclination, out of the pits after beginning facility usage.

10.1.2 Drainage tunnel

Since the underground facility is scheduled to be constructed at an altitude of approximately 110 m in the Tohoku ILC Civil Engineering Plan, drainage can be planned with gravitational flow by means of a drainage tunnel. Gravitational drainage through the drainage tunnel allows us to avoid full water damage of devices even in emergency cases such as long-term power failure.

Though examinations of the Global Design Effort do not include a drainage tunnel, the Tohoku ILC Civil Engineering Plan examines this tunnel with the aim of avoiding the full water damage of devices even in emergency cases such as long-term power failure.

10.1.3 Drainage system

Figure 10.1 shows a diagram of a drainage system after beginning facility usage.

Groundwater into the main linear accelerator tunnel is forced to be drained to the nearest access hall. Then, it is forced to be drained from the access hall to the ground surface through an access tunnel by a pump.

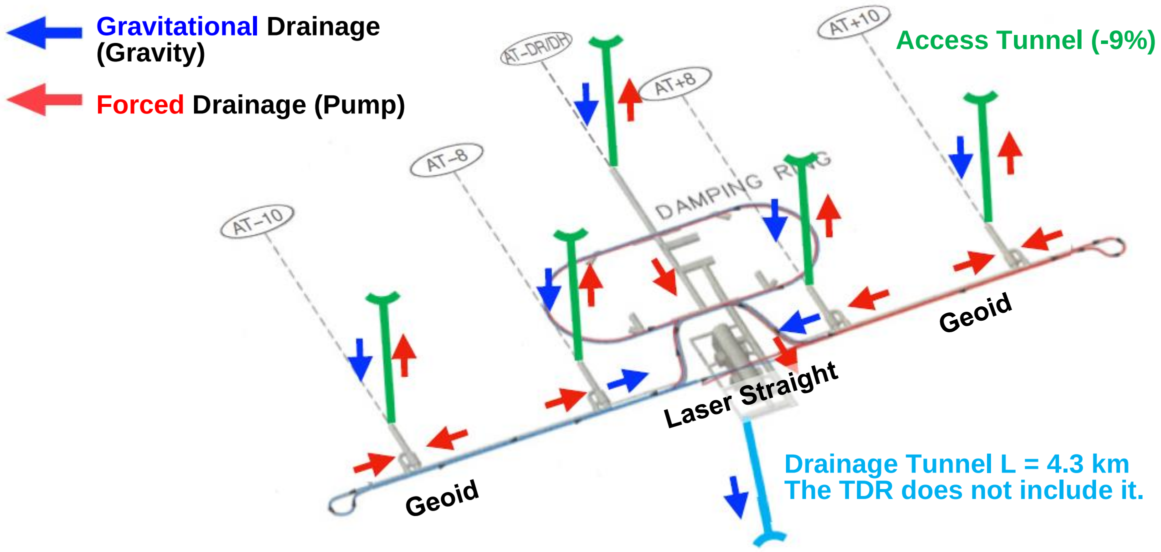


Figure 10.1. A diagram of the drainage system of the underground facility after the beginning of facility usage.

Groundwater into the BDS tunnel is flown to the detector hall by gravity, or drainage is forced. Then, the drainage is forced from the detector hall to the ground surface through an access tunnel by pump, or drained through a drainage tunnel by gravity.

10.2 Design specifications

10.2.1 Condition of constant groundwater inflow rate

We assumed the constant groundwater inflow rate after the beginning of facility usage to be “0.8 m^3/min/km” (see Chapter 4), estimated on the specific discharge of base flow and a groundwater-affected area.

10.2.2 Sectional drainage method in the accelerator tunnel

We assume the inclination necessary for drainage by gravitational flow to be 0.1% for the following considerations. This value corresponds to the lower limit of laying inclination standard values of 1/100 to 1/1,000 for agricultural culvert pipes [8].

To lead groundwater in a main linear accelerator tunnel to the nearest access hall, a section length of approximately 2.5 km (a half of the distance between adjacent access halls) is required. If groundwater for the maximum overall length of this section is drained by gravity, the difference of elevation necessary for ensuring an inclination of 0.1% is 2.5 km x 0.1% = 2.5 m, which is excessive for the central drainage pipe with a diameter of 300 mm under the bottom slab of the accelerator tunnel.

If the maximum overall length is shortened from 2.5 km to "250 m," the difference of elevation corresponding to an inclination of 0.1% becomes 250 mm, which makes construction realistic. Thus, we have determined the central drainage pipe to have an inclination of 0.1% alternately at 250m intervals, and make groundwater flow to drainage pits installed at 500-m intervals by gravity (see "Cross Section A-A" in Figure 10.2).

A vertical drainage ditch with an inclination of 0.1% is installed in the bottom slab of the accelerator tunnel, and drainage tanks are installed at 500-m intervals. Groundwater flown from drainage pits to drainage tanks is pumped up in the tanks by water pumps, and is then forced to flow to the nearest access halls while it is led successively (see "Cross Section B-B" in Figure 10.2).

Figure 10.2 shows a schematic drawing of the drainage system in the accelerator tunnel. Note that the rete of groundwater inflow into a section 500 m in length is assumed to be 0.8 m^3/min/km x 0.5 km x 130% = 0.52 m^3/min by considering an extra rate of 130% for agricultural drainage tunnels [9].

10.2.3 Dimensions and structures of the drainage tunnel

The drainage tunnel was designed according to the following conditions:

- Only one drainage tunnel is installed in the center of the underground facility.
- Drainage capabilities (an inclination and a cross section) are ensured to make groundwater in all pits flow by gravity in case of the long-term shut down of all power supply.
- The inner cross section is determined to be a "type-2r standard horseshoe-shaped cross section with the finished radius r = 2.7 m" because a "simultaneous work method of excavation and lining" is applied with consideration of the construction period of the drainage tunnel.
- For the risk of water damage, we have determined that "In case of power failure, it is acceptable for water level not to reach the height of beams (1.1 m above the bottom slab) in the accelerator tunnel."

The length of the drainage tunnel was determined to be 4,027 m. A storage tank (L50 m x W10 m x H2 m = 1,000 m^3) was determined to be installed on the pithead yard of the drainage tunnel, and a culvert 258 m in length was determined to be installed towards a river for discharging. The overall length is 4,335 m.

Figure 10.3 shows an overall structural drawing of the drainage tunnel.

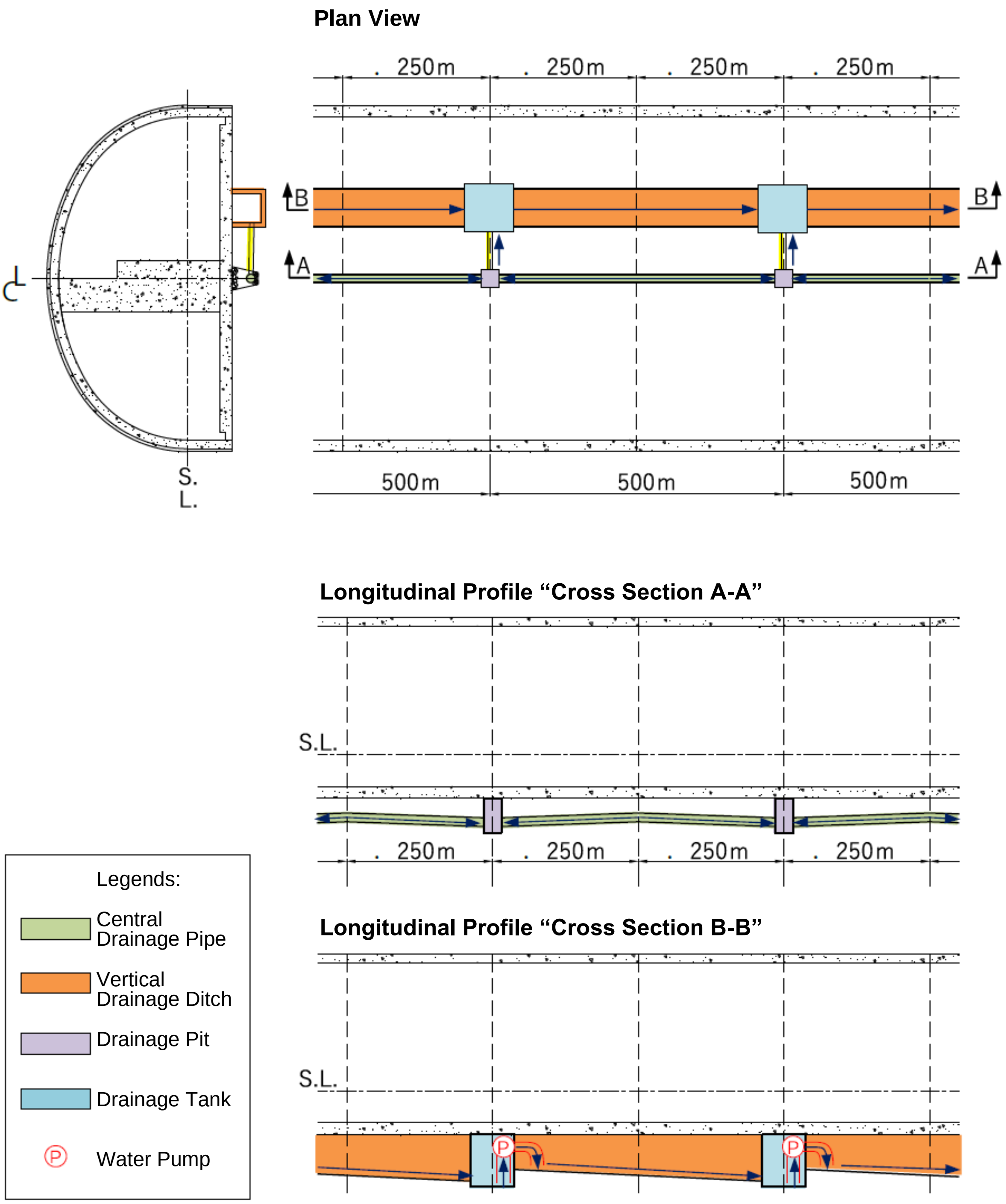


Figure 10.2. A schematic drawing of the sectional drainage system in the accelerator tunnel.

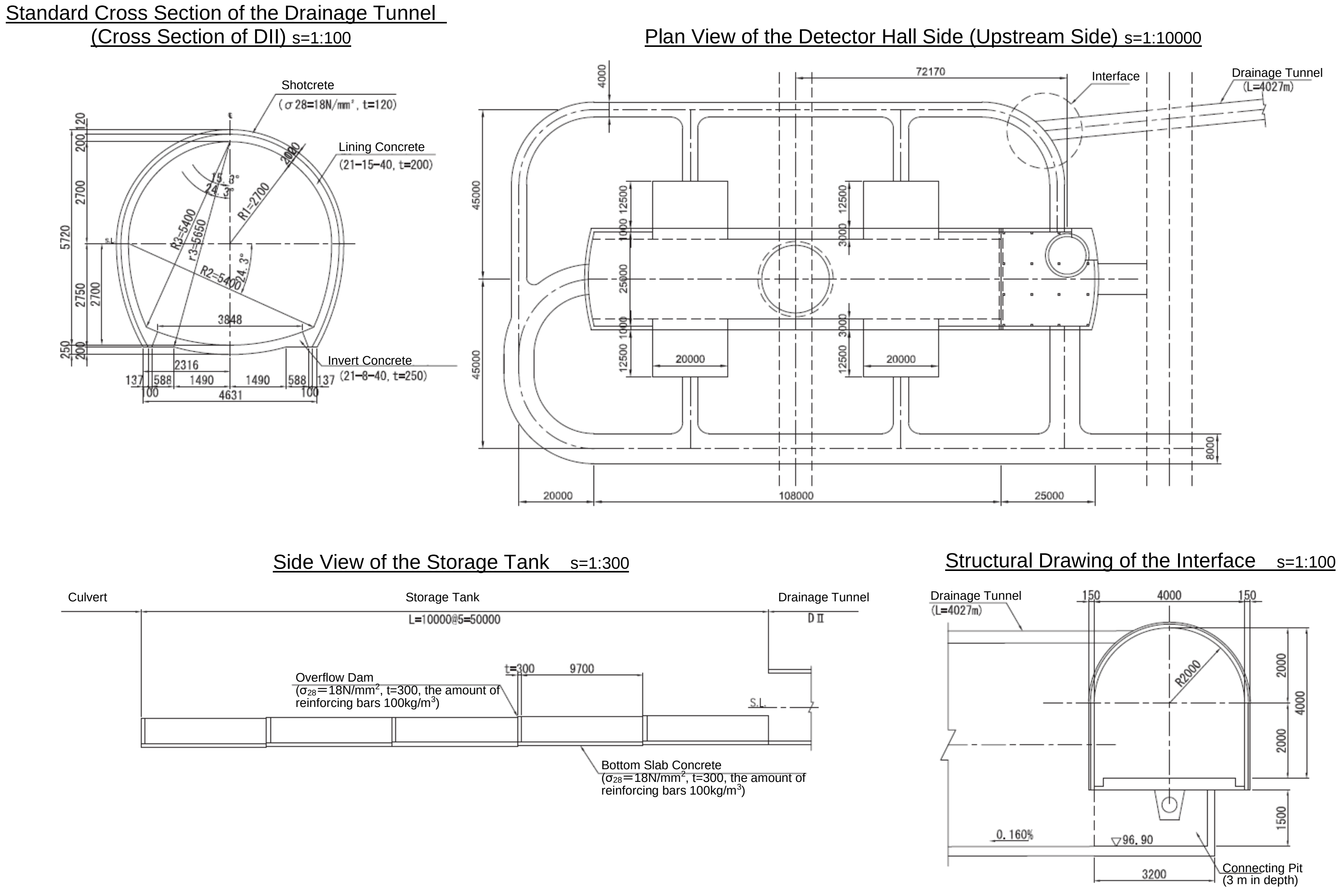


Figure 10.3. An overall structural drawing of the drainage tunnel.

Support patterns of the drainage tunnel were determined to be as follows: pattern pithead-DII for sections with an overburden of 2D (12.08 m) or less; pattern DI for those with an overburden of 2D to 30 m; pattern CI for those 147 m in length near the accelerator tunnel; and pattern CII for other sections.

Support and lining structures were determined as Table 10.1 in accordance with standard support patterns for agricultural drainage tunnels. Figures 10.4 to 10.7 show standard cross-sectional views and support patterns of rock mass classes.

Table 10.1. Support and lining structures of the drainage tunnel

<table>
<tr><th>Rock mass classes</th><th>One digging length</th><th>Shotcrete</th><th>Rock bolt</th><th>Steel support</th><th>Lining concrete</th><th>Invert concrete</th></tr>
<tr><td>CI</td><td>1.5 m</td><td>A thickness of 5 cm
(18 N/mm^2)</td><td>A length of 2 m
1.2-m intervals in the circumferential direction
1.5-m intervals in the longitudinal direction
A yield strength of 117.7 kN</td><td>–</td><td>A thickness of 20 cm
(18 N/mm^2)</td><td>A thickness of 20 cm
(18 N/mm^2)</td></tr>
<tr><td>CII</td><td>1.2 m</td><td>A thickness of 5 cm
(18 N/mm^2)</td><td>A length of 2 m
1.2-m intervals in the circumferential direction
1.2-m intervals in the longitudinal direction
A yield strength of 117.7 kN</td><td>–</td><td>A thickness of 20 cm
(18 N/mm^2)</td><td>A thickness of 25 cm
(18 N/mm^2)</td></tr>
<tr><td>DI</td><td>1.0 m</td><td>A thickness of 10 cm
(18 N/mm^2)

The entire circumference of a wire net</td><td>A length of 2 m
1.0-m intervals in the circumferential direction
1.0-m intervals in the longitudinal direction
A yield strength of 117.7 kN</td><td>H-100×100 ×6×8 @1.0m</td><td>A thickness of 20 cm
(18 N/mm^2)</td><td>A thickness of 25 cm
(18 N/mm^2)</td></tr>
<tr><td rowspan="2">DII</td><td rowspan="2">1.0 m</td><td rowspan="2">A thickness of 12 cm
(18 N/mm^2)

The entire circumference of a wire net</td><td>(Excluding 120° of the crown)
A length of 3 m
1.0-m intervals in the circumferential direction
1.0-m intervals in the longitudinal direction
A yield strength of 117.7 kN</td><td rowspan="2">H-100×100 ×6×8 @1.0m</td><td rowspan="2">A thickness of 20 cm
(21 N/mm^2)</td><td rowspan="2">A thickness of 25 cm
(21 N/mm^2)</td></tr>
<tr><td>(120° of the crown)
“Filling-type FP” A length of 2 m
0.5-m intervals in the circumferential direction
1.0-m intervals in the longitudinal direction
A yield strength of 117.7 kN</td></tr>
</table>

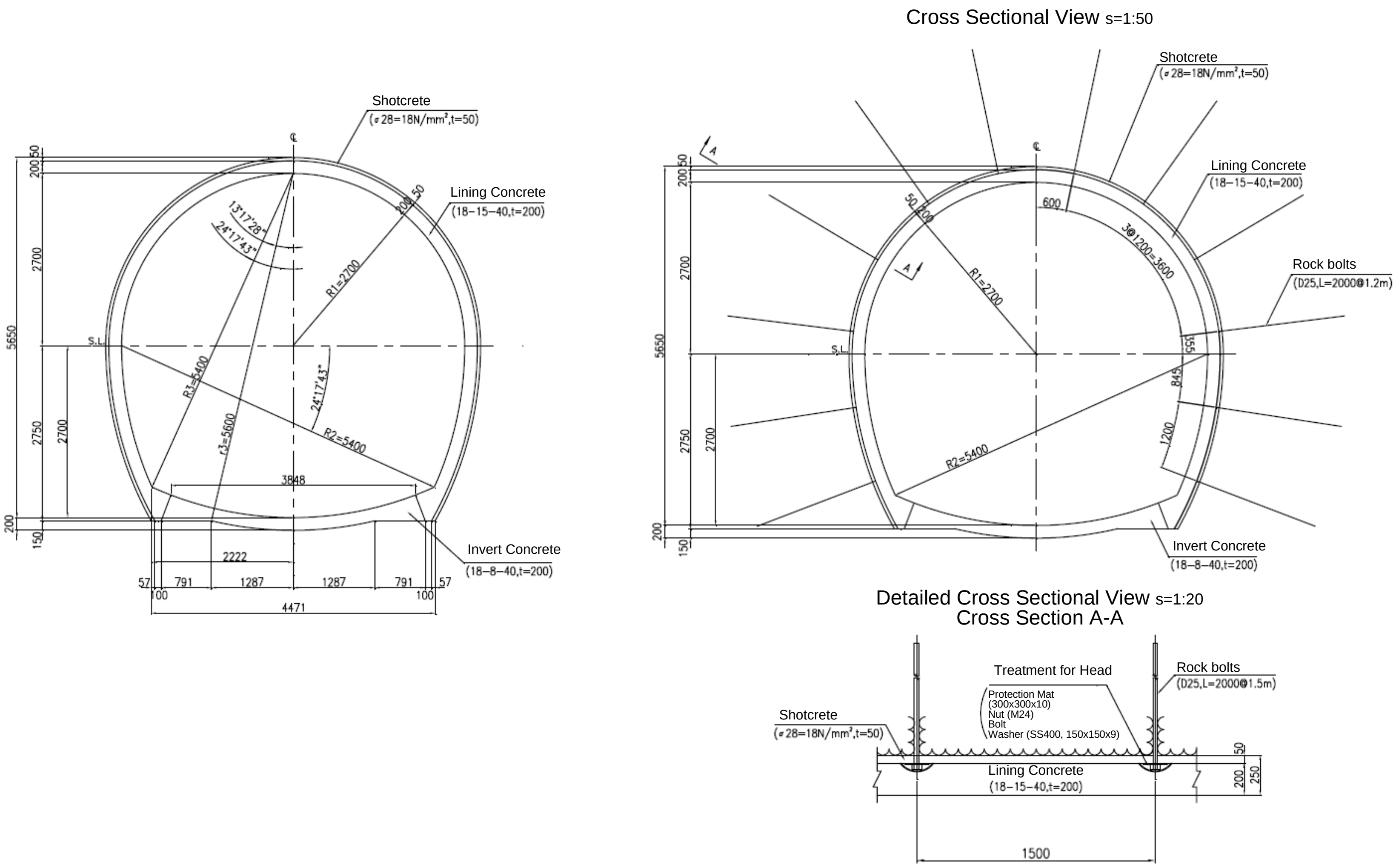


Figure 10.4. Standard cross-sectional view (left) and support pattern (right) of class CI.

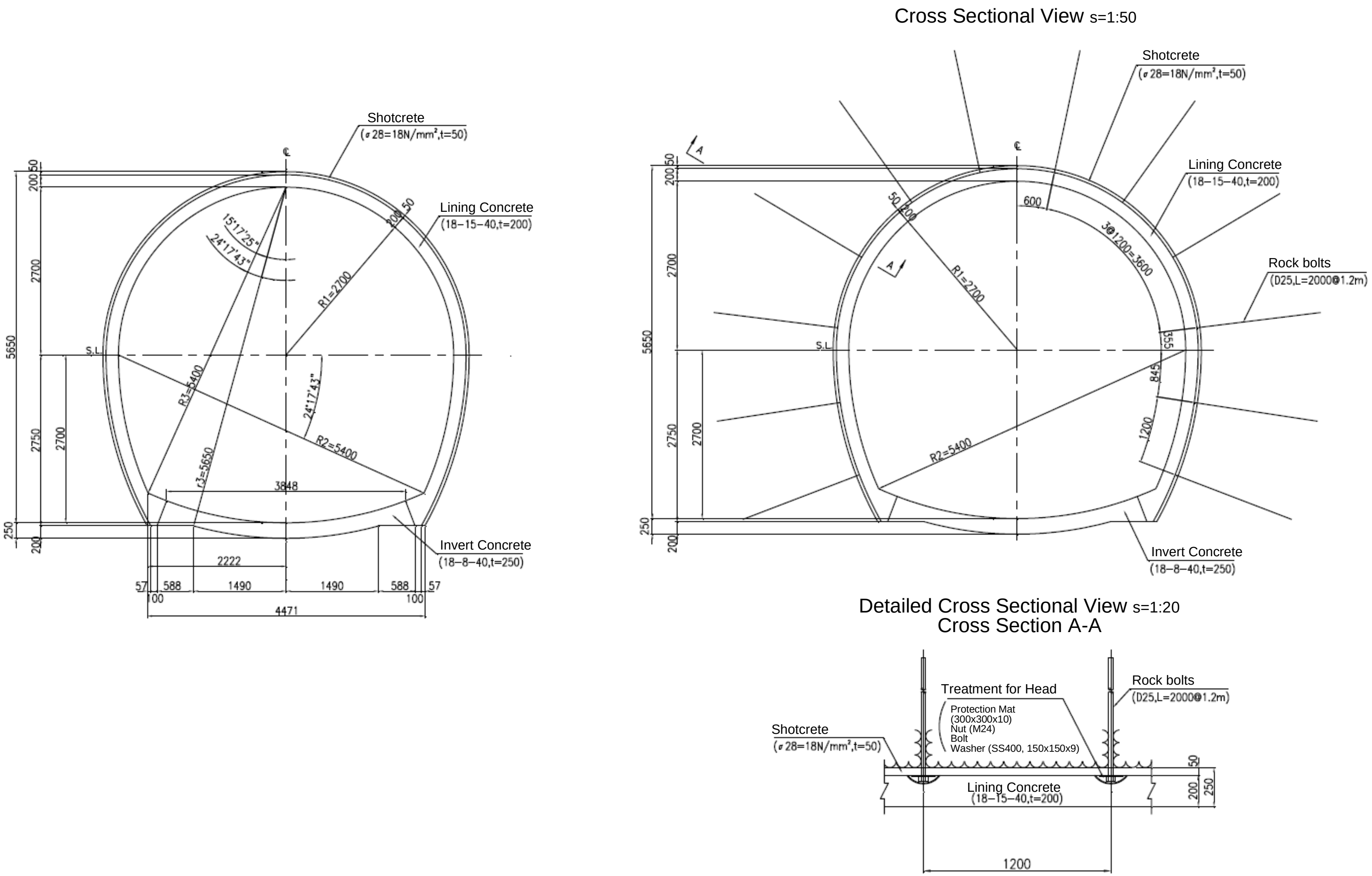


Figure 10.5. Standard cross-sectional view (left) and support pattern (right) of class CII.

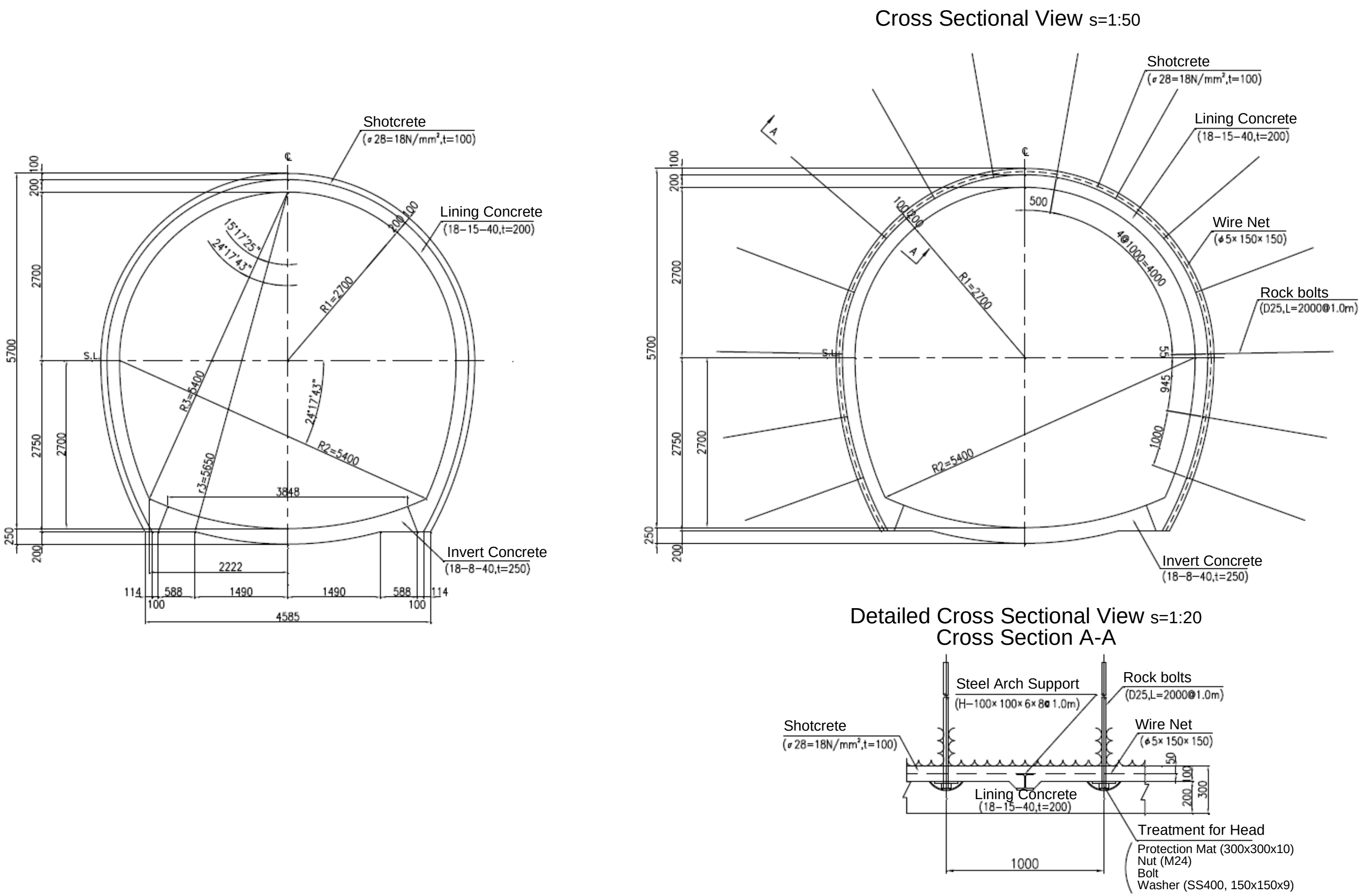


Figure 10.6. Standard cross-sectional view (left) and support pattern (right) of class DI.

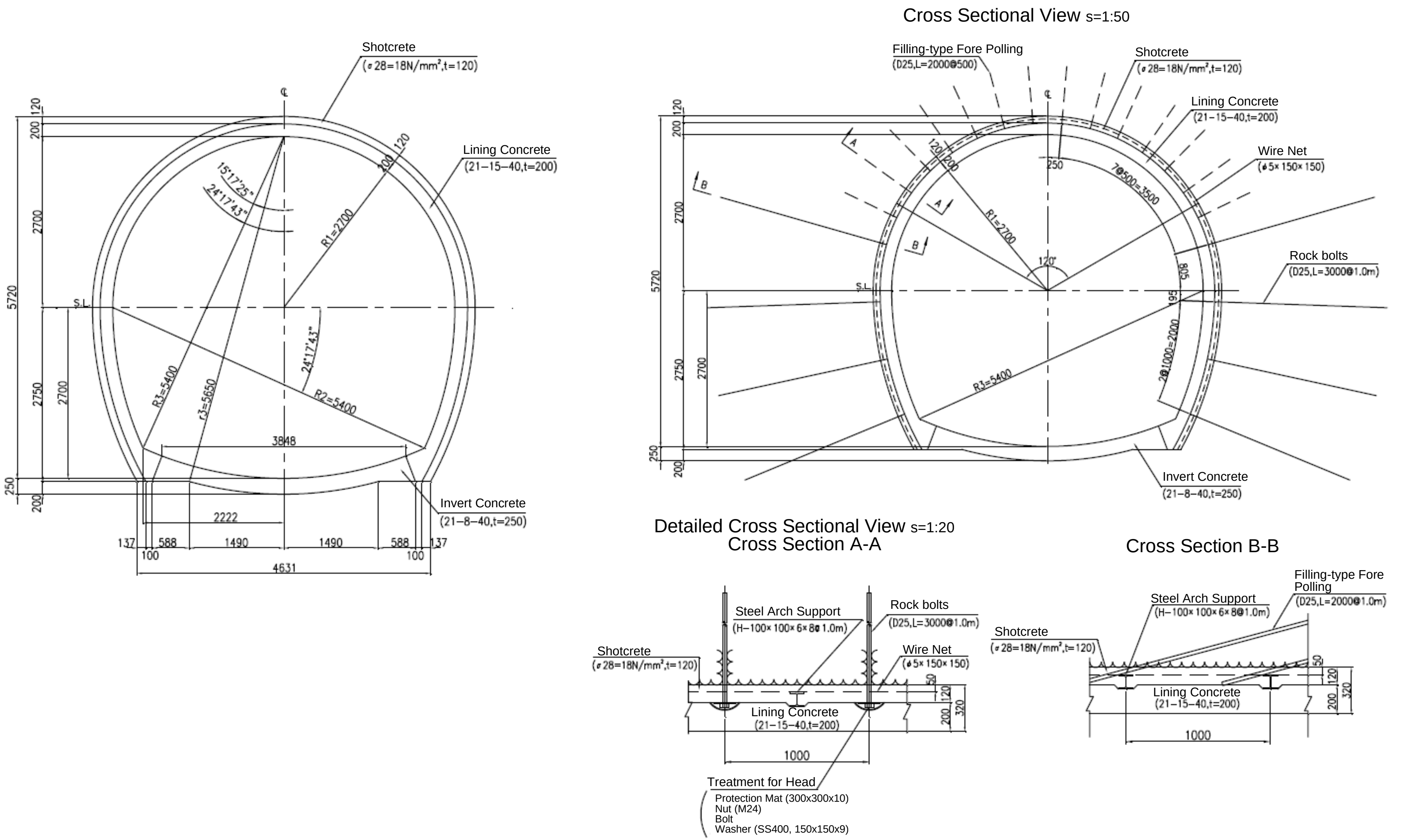


Figure 10.7. Standard cross-sectional view (left) and support pattern (right) of class DII.

10.3 Construction plan

10.3.1 Outline of the construction

Construction methods were according to the following conditions:

- A blasting excavation method, a full-face technique, is applied.
- Concepts of pay lines (for outbreak, extra shot, and extra lining) are in accordance with the estimation standards.
- Standards on excavating equipment (drill carriages, muck loaders, muck cars, concrete guns, agitator cars, battery-driven locomotives, and other vehicles) are in accordance with the estimation standards.
- For the mucking method, a (double-tracked) rail method is applied because the designed excavation cross section of the drainage tunnel is as small as 20 to 35 m^2.
- The excavation muck is assumed to be placed near a pithead temporarily, and then transported secondarily to a place 10 km away from the pithead by 10-ton dump trucks for disposal. Since the appearance of mineralized muck is assumed to be less likely according to the results of geological surveys, we do not consider the costs and facilities for mineralized muck disposal. If mineralized muck appears during construction, it is disposed of appropriately.
- Types and standards of in-pit and pithead yard temporary facilities are in accordance with the estimation standards; however, a surplus of over 1,000 m is included appropriately according to length.

10.3.2 Outline of the construction process plan

Figure 10.8 shows the overall processes for the drainage tunnel. The construction period was estimated at 88.5 months.

In addition, the present construction process plan does not include the following activities: the new installation of roads from ordinary roads to pithead yards for construction; the widening and reinforcement of existing roads; and the rebuilding and new installation of bridges. This is because these are assumed to be conducted in advance with other construction work as necessary. Similarly, the present construction process plan does not include the periods necessary for obtaining land for pithead yards, roads for construction, surplus soil disposal facilities, and other purposes.

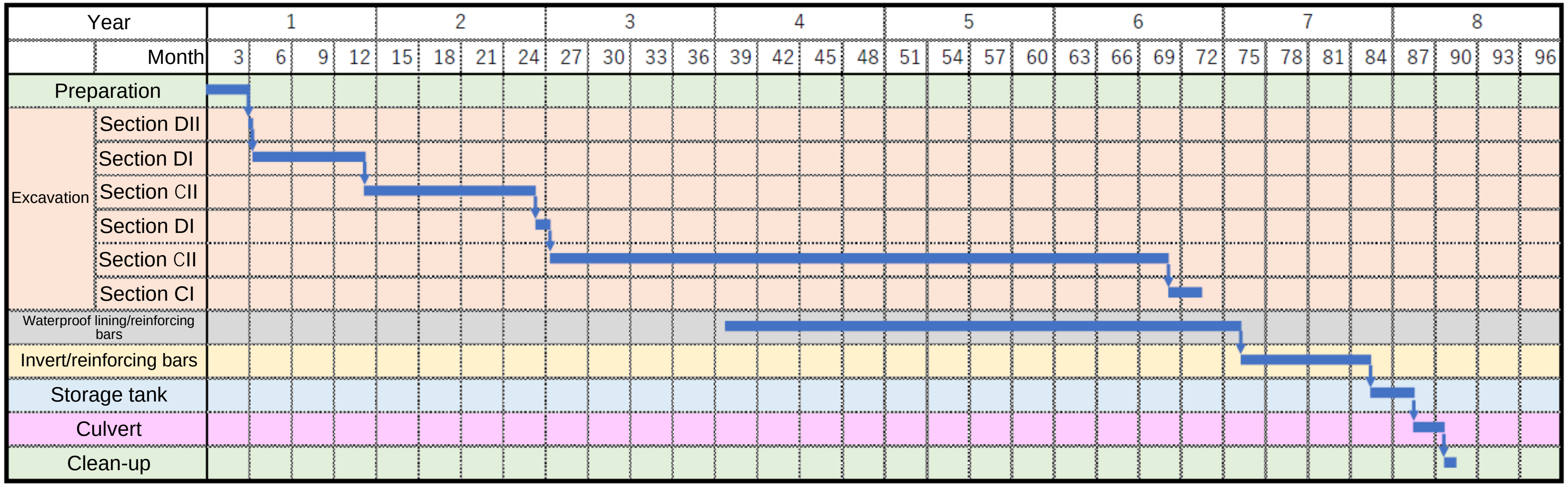


Figure 10.8. Processes for the drainage tunnel.

11 Outline of the Construction Plan

Figure 11.1 shows a table of the outlined overall processes. Construction period was estimated at 90.8 months.

The construction period is determined by an excavation section starting from AS-10 towards a loop section at an end. In this excavation section, the construction of a main linear accelerator tunnel starts most lately due to the longest length of access tunnel AT-10. Since the end has no access tunnels, in addition, one-way tunneling is to be conducted in the section starting from access hall AH-10 towards the loop section; thus, methods for shortening the construction period are limited.

Though the drainage tunnel has as long a length as 4.3 km and also has a long construction period, it is to be completed before the beginning of facility usage.

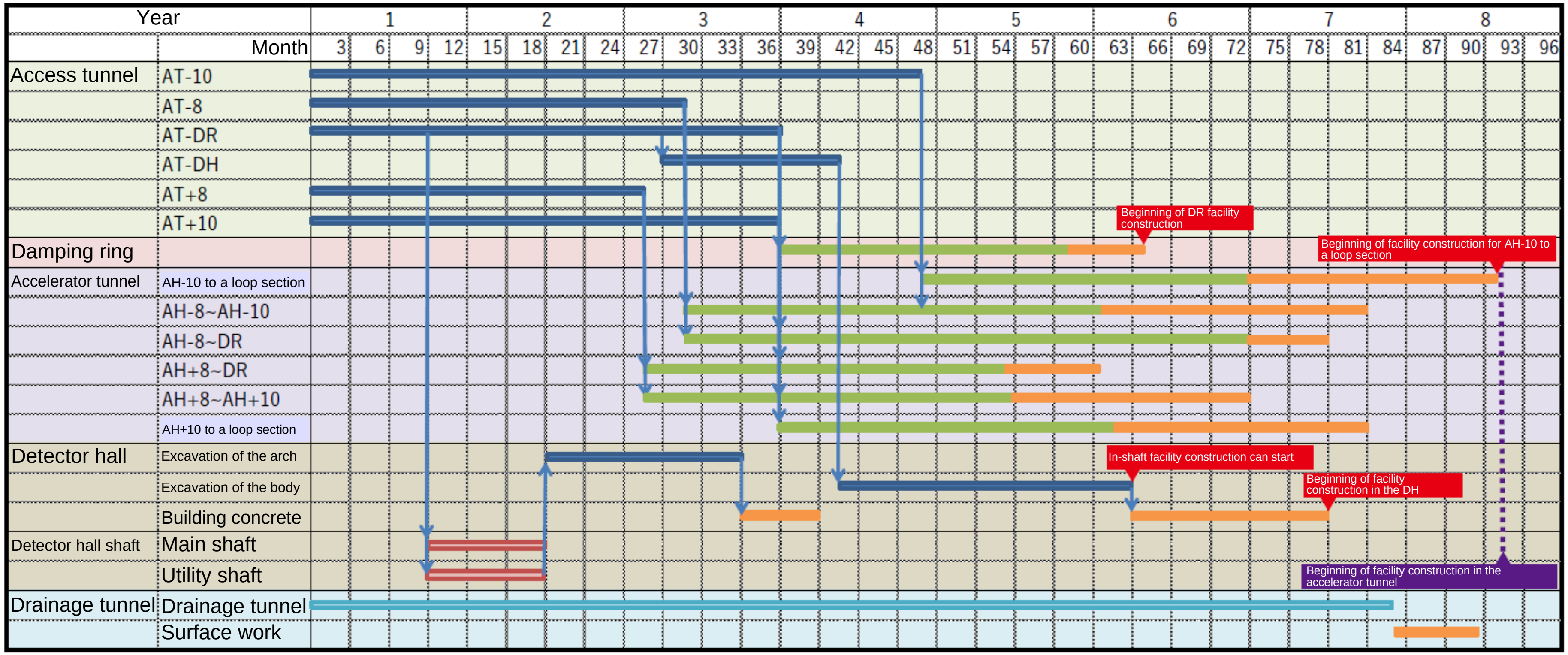


Figure 11.1. Table of the outlined overall processes.

12 Approximate Construction Costs

Table 12.1 shows the approximate construction costs. These construction costs were estimated on the basis of equipment ownership costs, material unit costs, and labor unit costs disclosed in the 2014 fiscal year.

This estimation was conducted for the civil engineering plan settled by the Tohoku ILC Preparation Office with consideration of the conditions (topography, geology, and other features) intrinsic to the Kitakami site by creating a construction plan in accordance with the Civil Engineering Estimation Standards (the 2014 fiscal year edition) of the Ministry of Land, Infrastructure, Transport, and Tourism.

We consider that the construction plan does not require any major changes from the 2014 fiscal year onward; since unit costs vary every fiscal year, however, the results of estimates are also expected to vary.

Table 12.1. Approximate construction costs (unit: 100 million yen)

Part	An overall length of 20.5 km	
	Specifications	Construction price
Access tunnels	AT: 5 tunnels （5,662 m in length） AH: 5 halls (248,354 m^3) Preparation of AS yards: 5 places x 18,193 m^2	247
Accelerator tunnel	Semicylindrical shape (20.5 km in length) Partition walls: Made on the site (15.3 km in length) DR, end loop sections, BDS (beam and service), and other components	644
Detector hall	Large cavern, shafts, and peripheral tunnels Same pithead of AT-DH/AT-DR Preparation of yards: 78,500 m^2	134
Drainage facilities	A tunnel, a storage tank, and a culvert (4,335 m in overall length) In-pit cross water drainage and out-pit facilities	73 30
Total		1,127

13 Challenges for the Future

The Global Design Effort is conducting detailed examinations for the periphery of the positron source, beam dump, detector hall, and other components. We will continue coordination in response to the progress of these examinations in the future.

We consider that other detailed geological surveys and design examinations are required for a detailed design, mainly in the following places:

- Cross points of rivers
- Pitheads
- Cross areas of the crown of the detector hall and the shafts

The result of the geological surveys shows that the cross points of rivers have worse geological conditions in comparison with other places. Since these points are limited, however, we have not conducted detailed examinations for them so far. Construction methods are required to be examined in detail for places with a thin overburden and rivers directly above them. Further examination of influences on the rivers is also required.

Pitheads on the ground surface generally have poor geological conditions and conditions are expected to be varying. We consider that the pitheads require detailed geological surveys by the time design for implementation is conducted.

We consider that the detector hall and shafts have good stability because of good rock mass conditions. On the other hand, we consider that the cross areas of the crown (the roof part) of the detector hall and shafts require other detailed examinations for their strength and stability.

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[4] Added to the Seamless Digital Map of Japan (1:200,000) (by the Geological Survery of Japan, the National Institute of Advanced Industrial Science and Technology).

[5] Hikoharu TAKAHASHI. "Applied Geological Examination for Tunnel Spring Water." *Railway Technical Research Report*. 1962, No. 279, pp. 1-103.

[6] Hiroshi OSHIMA and Shinichi NISHIMORI. "Research on Hydrologic Survey Methods for Tunnel Construction." *Railway Technical Research Report*. 1979, No. 1108, pp. 1-186.

[7] Kenichi FUMIMURA and Michito SHIMO. "Development of GIS System for Predicting Tunnel Inflow and Dewartered Area." *Papers/Reports of Tunnel Engineering Research Presentations*. 2003, 13, pp. 327-332.

[8] *Design and Construction of Underdrainage*. The Agricultural Upland Development Association (a general incorporated association). Mar., 1982, pp. 62.

[9] *Standards and Technical Instructions for "Aqueduct Tunnels" in Land Improvement Activity Plan Design and Standard Design*. The Structural Improvement Bureau, the Ministry of Agriculture, Forestry, and Fisheries. Oct., 1996.

Part IV

Appendix

A Other Facility Location Plans

Since the Kitakami site has relatively gentle topography, multiple facility location plans for the site can be created easily. The underground facility changes little even though the facility location changes; however, the lengths of the access tunnels change depending on topographical conditions. Table A.1 now lists the lengths of the access tunnels for another two plans in which the location of the detector hall is different. Plan A of detector hall location is detailed in this document. Plan B is that the detector hall is moved 180 m to the west. Plan C is a facility location plan in which the detector hall is placed to the north of Okita River, as introduced in Chapter 6. The comparison of these three plans shows that the variation of the total length of the access tunnels is within approximately 20%.

These location plans have different features. We can examine them in terms of various aspects including the total length of the access tunnels after the details of the ILC accelerator facility are finally decided.

Table A.1. List of the lengths of the access tunnels

Entry point (pithead)	Location of the detector hall		
	Plan A	Plan B	Plan C
AS−10	1,707 m	1,503 m	1,990 m
AS− 8	817 m	691 m	1,560 m
AS−DH	747 m	693 m	1,075 m
AS−DR	817 m	763 m	560 m
AS+ 8	427 m	283 m	935 m
AS+10	1,147 m	943 m	715 m
Total length	5,662 m	4,876 m	6,835 m

B Present Examination of the Main Beam Dump Cavern (for Reference)

We are examining a cavern containing the main beam dump. Drawings for the examination are shown from the next page onward.

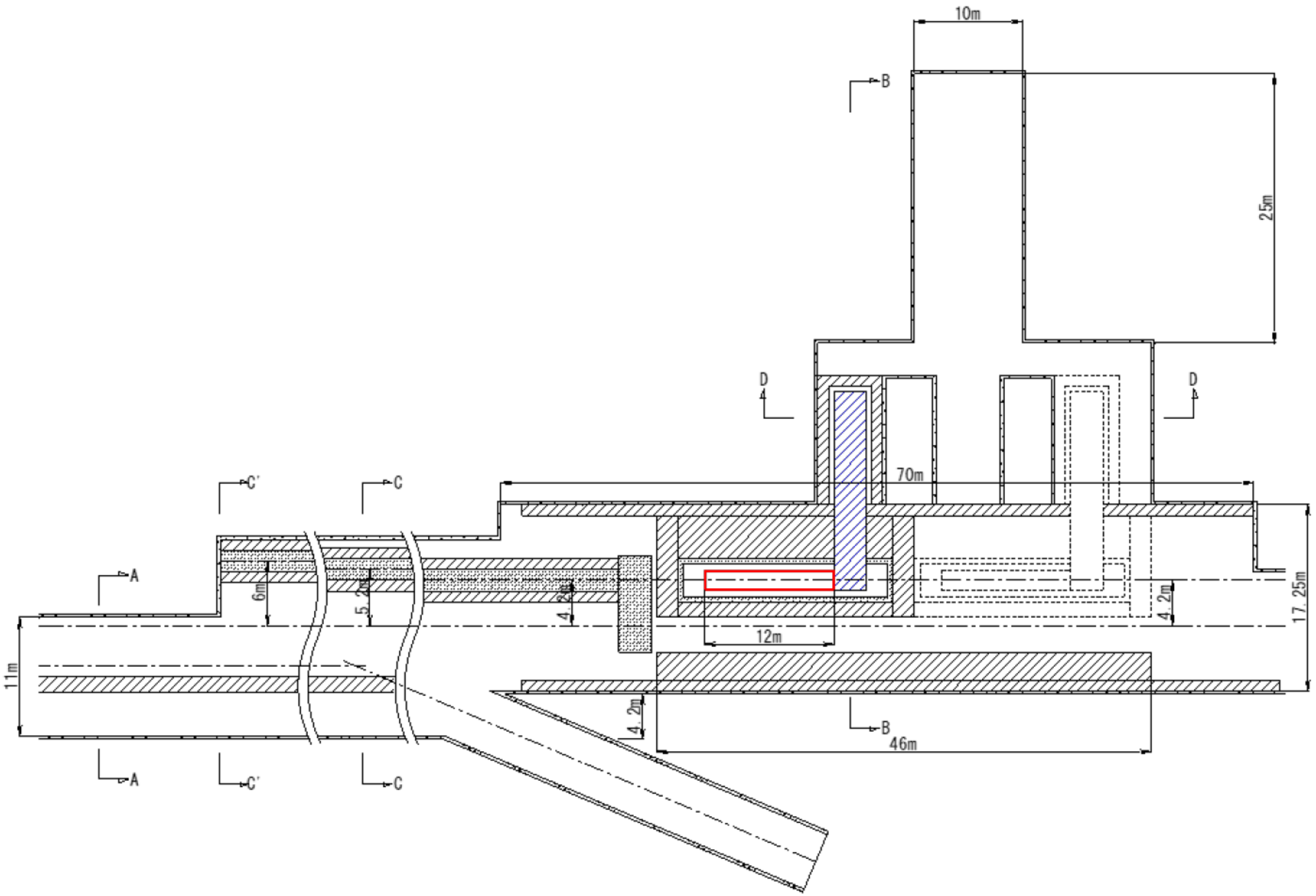


Figure B.1. Plan view of the periphery of the main beam dump cavern.

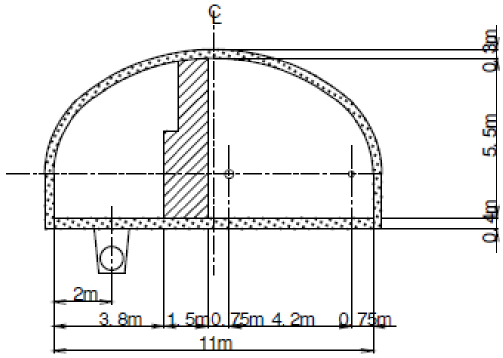


Figure B.2. Cross section A-A of the main beam dump cavern.

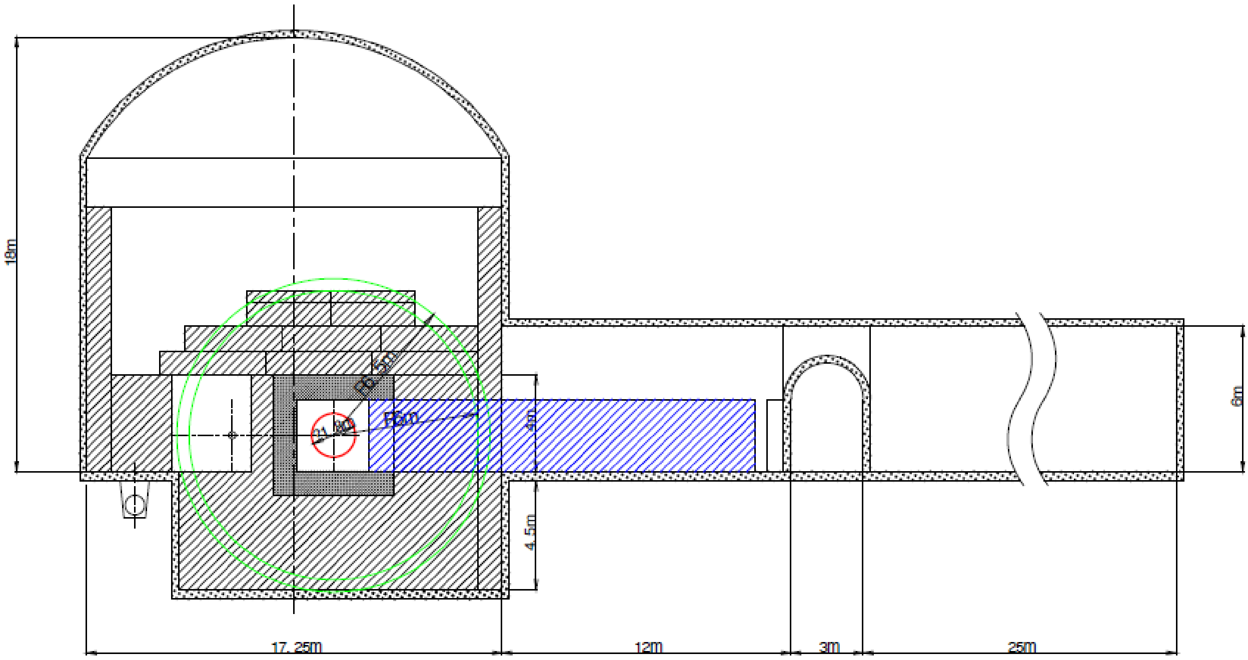


Figure B.3. Cross section B-B of the main beam dump cavern.

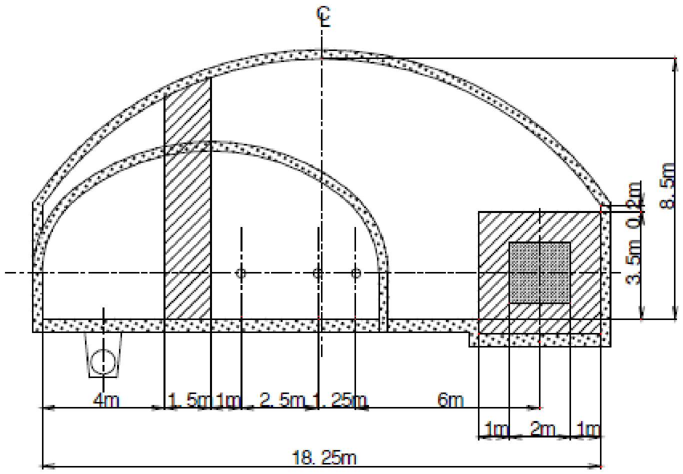


Figure B.4. Cross section C-C of the main beam dump cavern.

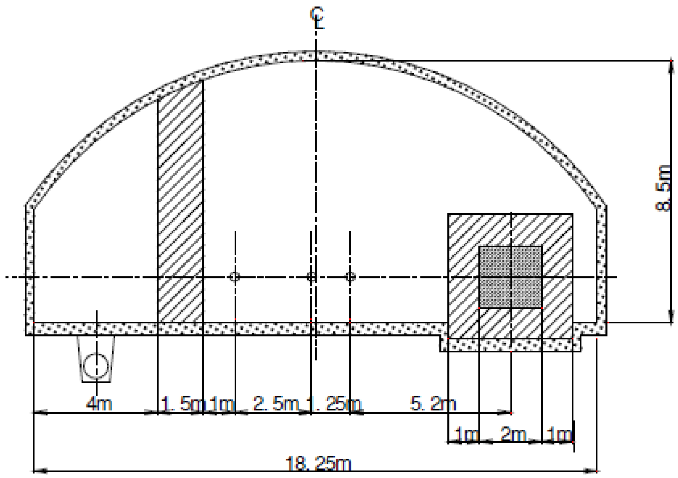


Figure B.5. Cross section C'-C' of the main beam dump cavern.

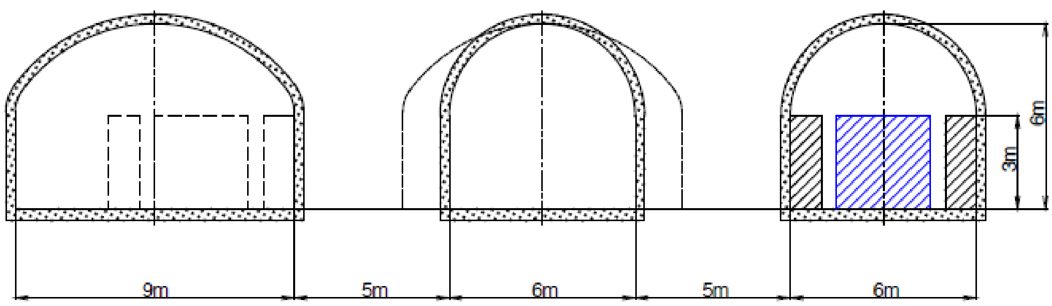
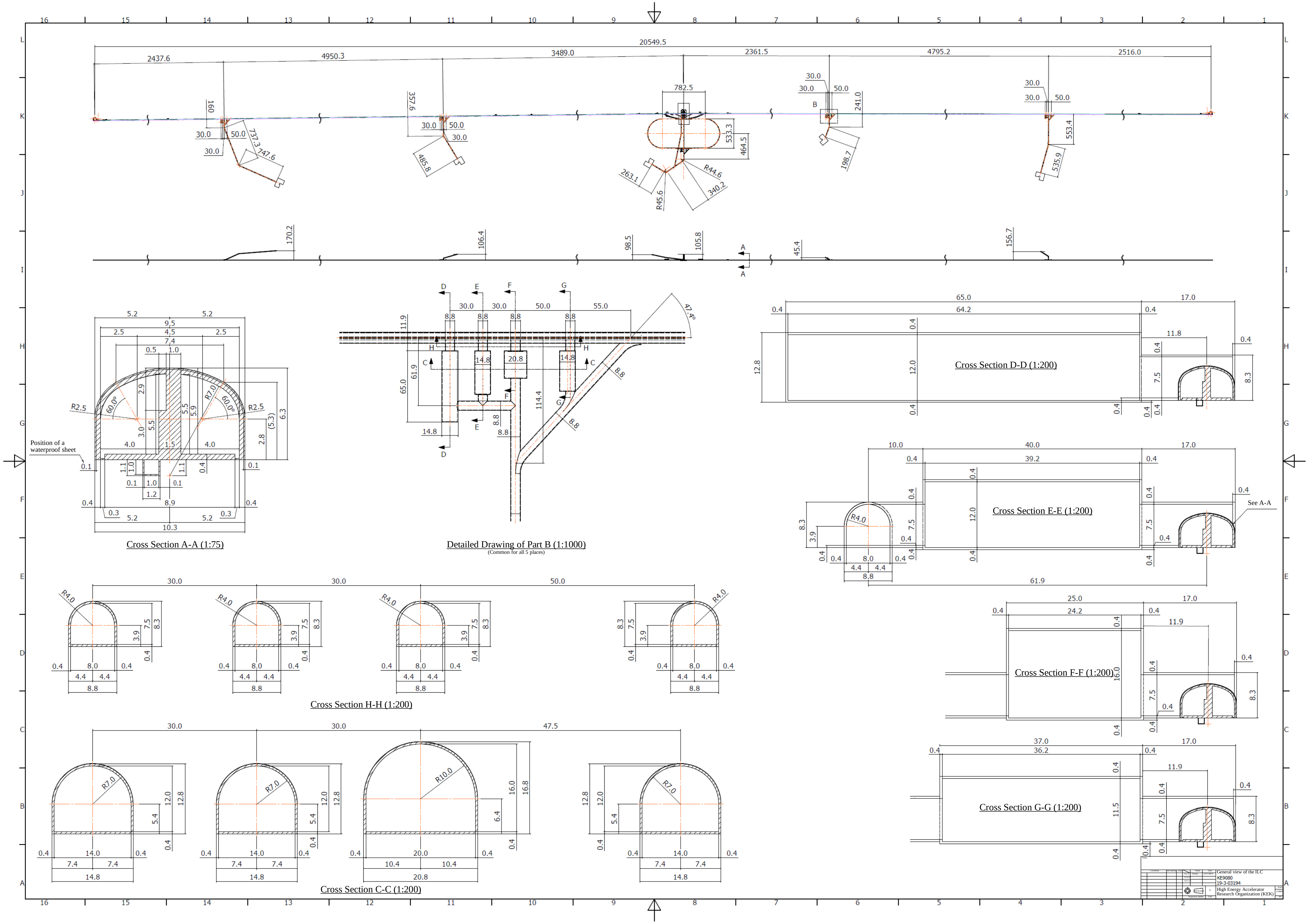
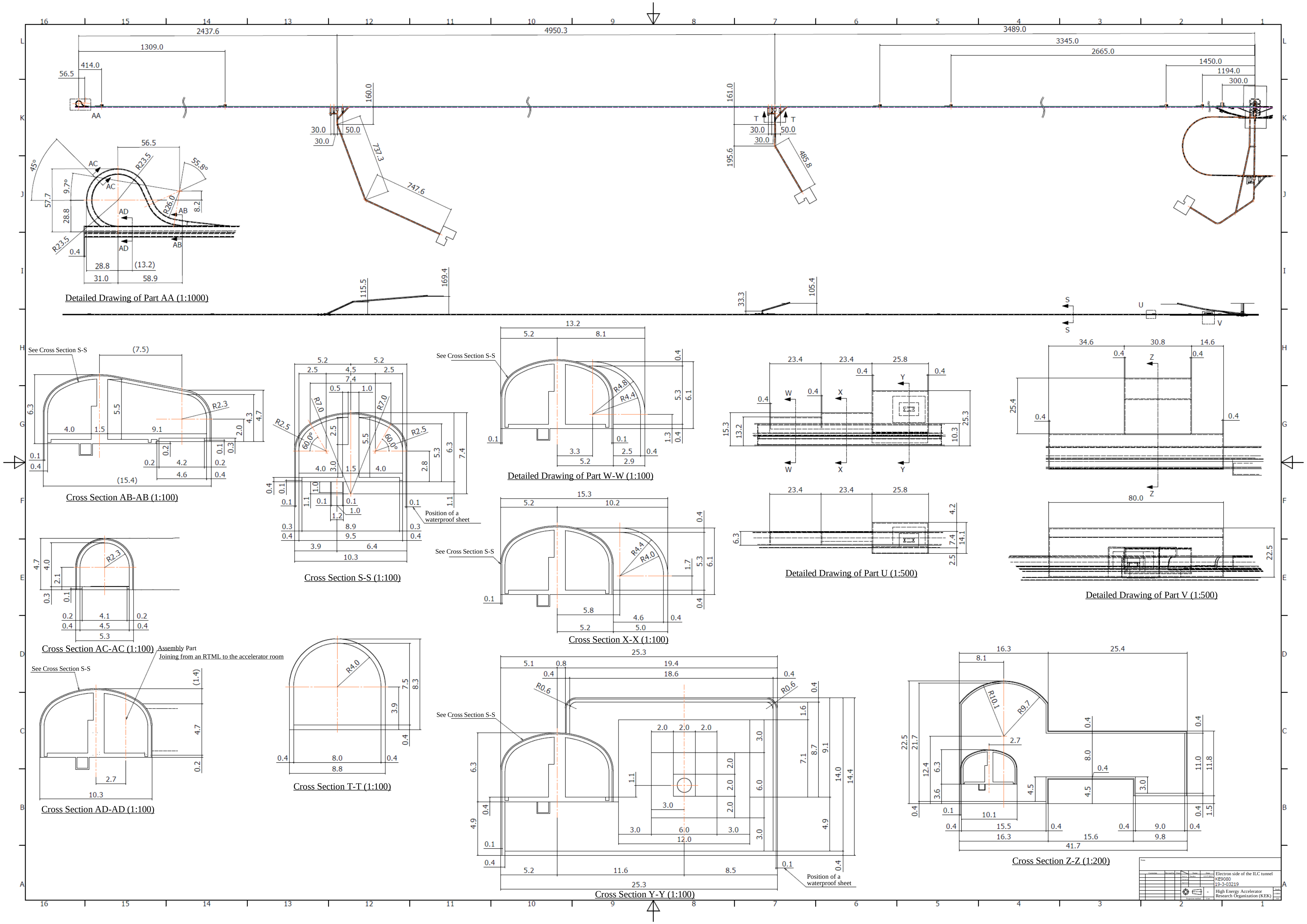


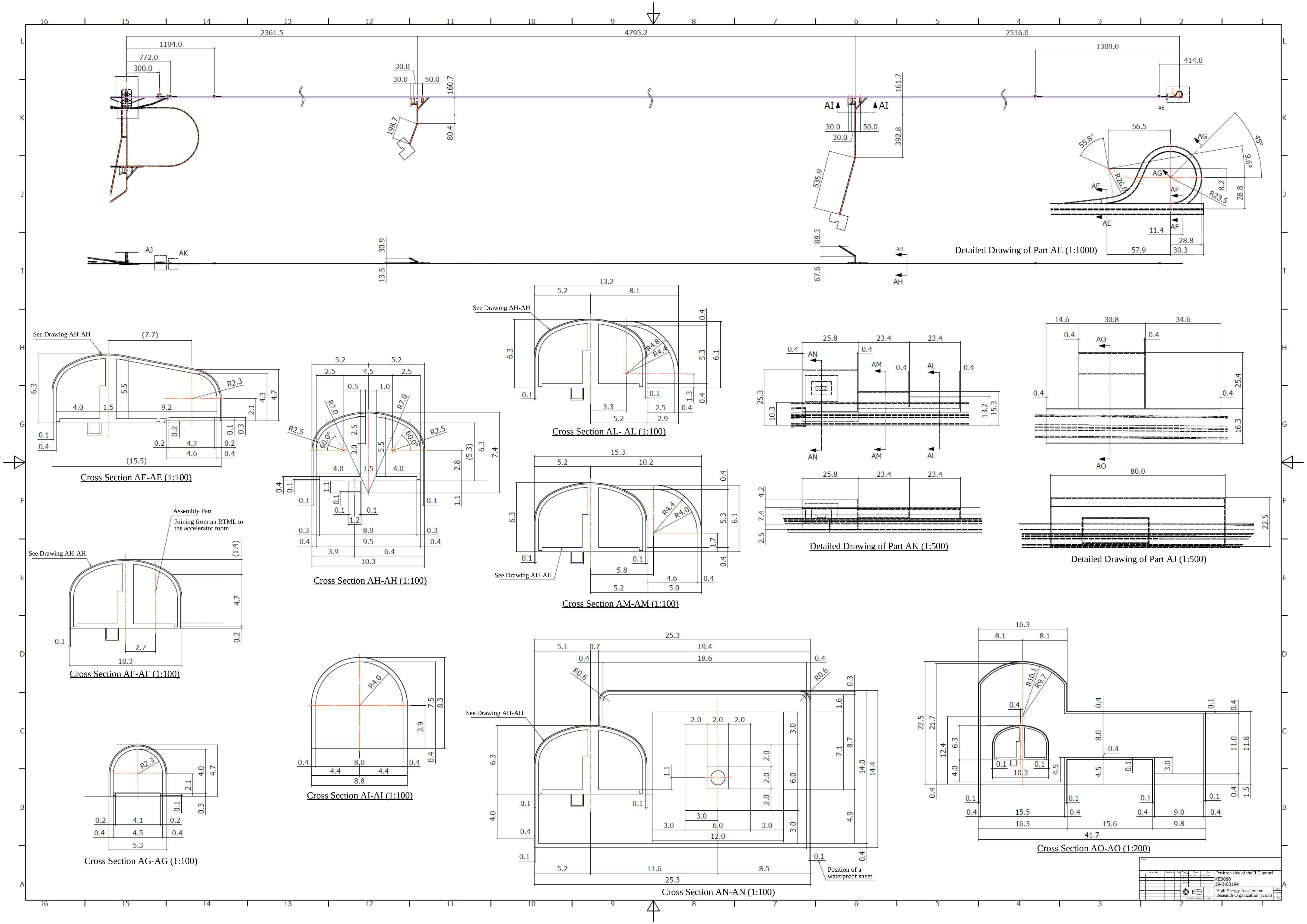
Figure B.6. Cross section D-D of the main beam dump cavern.

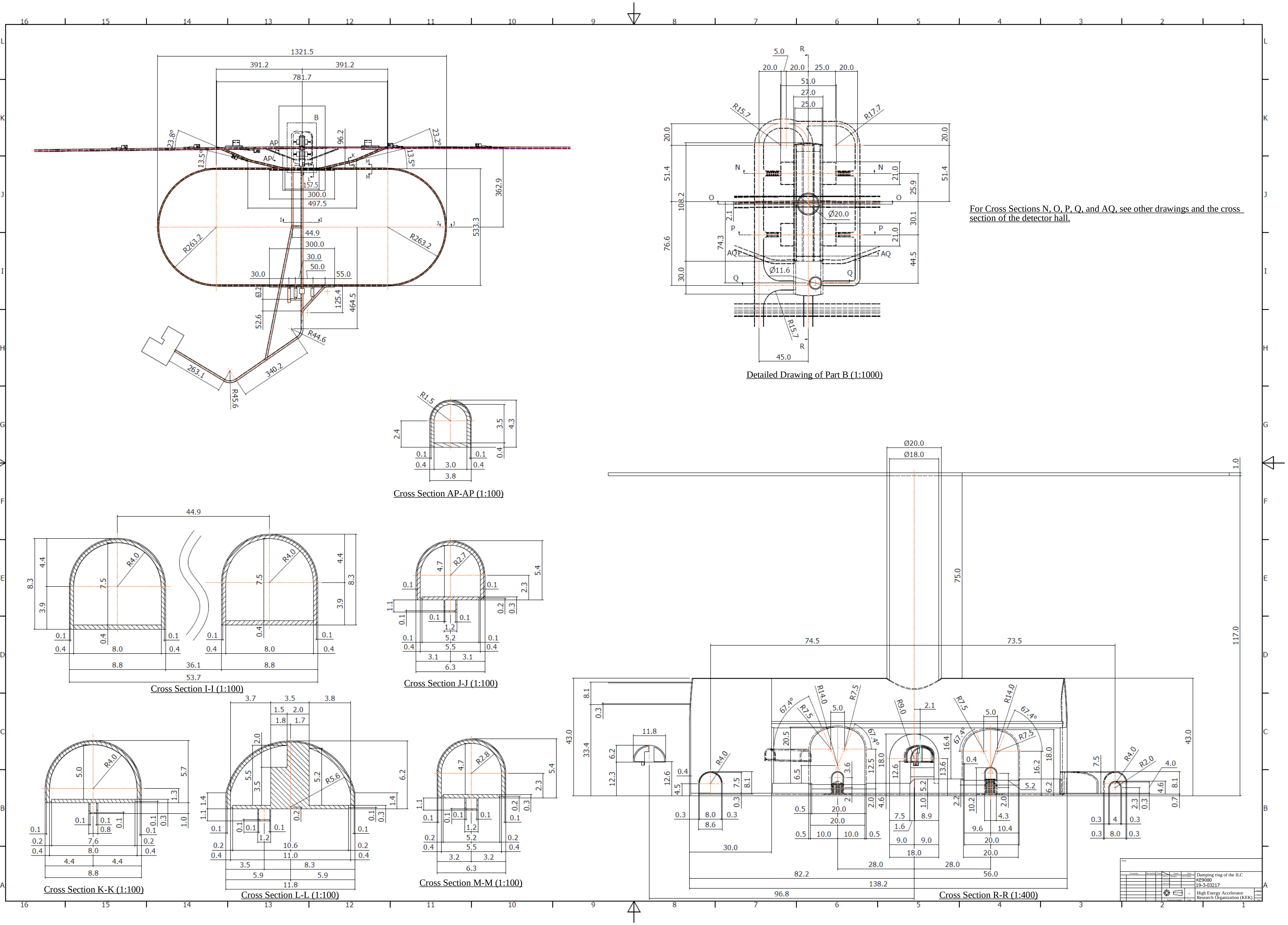
C Examination Drawings (for Reference)

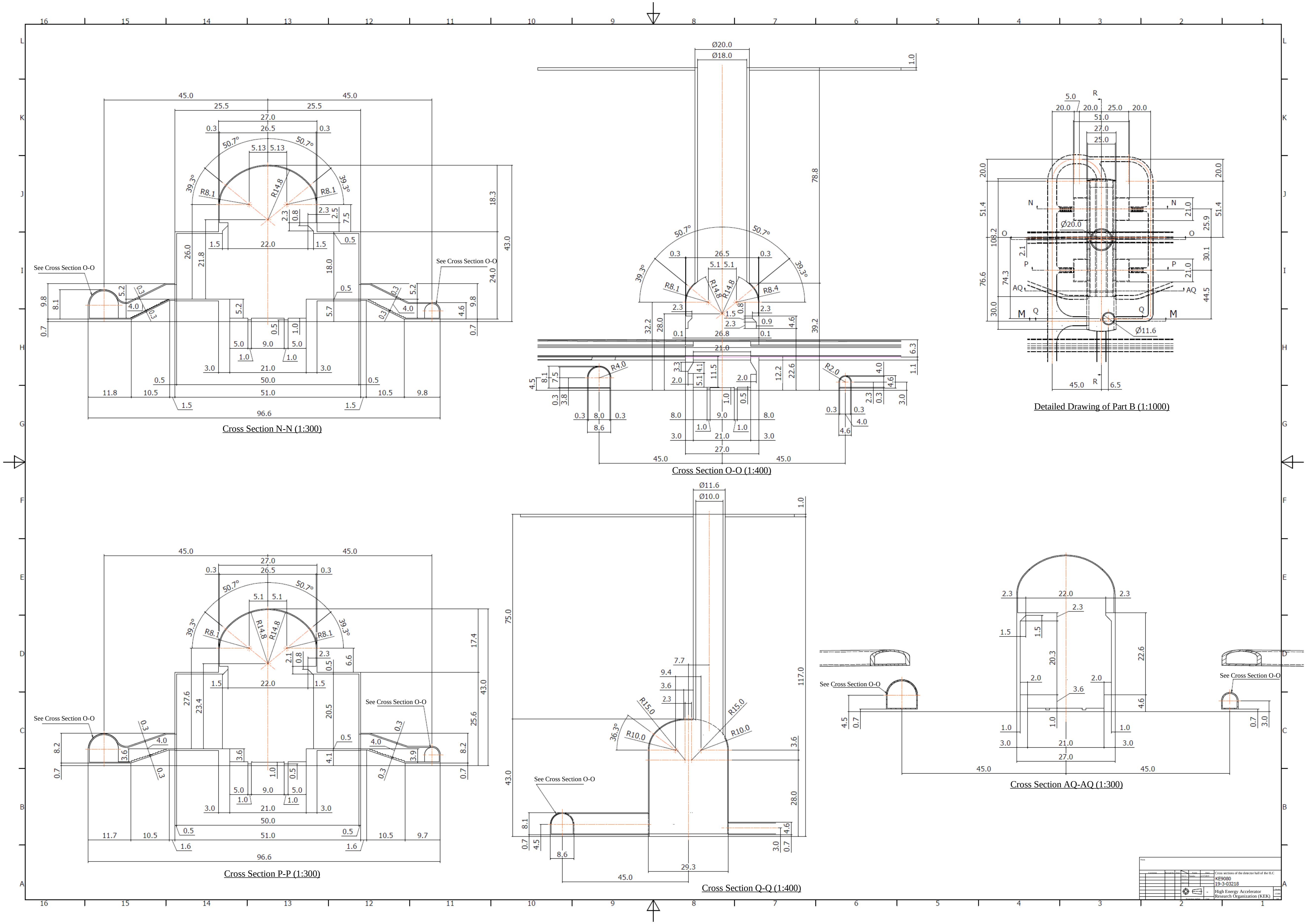
We are organizing drawings for examination with the High Energy Accelerator Research Organization as the central figure. Drawings as of January 2020 are shown from the next page onward. Though the Tohoku ILC Civil Engineering Plan was examined according to drawings as of 2018, no major changes have been conducted up to now.


Cross Section A-A (1:75)
Position of a waterproof sheet
Detailed Drawing of Part B (1:1000)
(Common for all 5 places)
Cross Section D-D (1:200)
Cross Section E-E (1:200)
See A-A
Cross Section F-F (1:200)
Cross Section G-G (1:200)
Cross Section H-H (1:200)
Cross Section C-C (1:200)
General view of the ILC
KE9080
19-3-03194
High Energy Accelerator Research Organization (KEK)


Detailed Drawing of Part AA (1:1000)
Cross Section AB-AB (1:100)
Cross Section S-S (1:100)
Detailed Drawing of Part W-W (1:100)
Detailed Drawing of Part U (1:500)
Detailed Drawing of Part V (1:500)
Cross Section AC-AC (1:100)
Assembly Part
Joining from an RTML to the accelerator room
Cross Section X-X (1:100)
Cross Section AD-AD (1:100)
Cross Section T-T (1:100)
Cross Section Z-Z (1:200)
Cross Section Y-Y (1:100)
See Cross Section S-S
Position of a waterproof sheet
Electron side of the ILC tunnel
KE9080
19-3-03219
High Energy Accelerator Research Organization (KEK)


Detailed Drawing of Part AE (1:1000)
Cross Section AE-AE (1:100)
Cross Section AH-AH (1:100)
Cross Section AL- AL (1:100)
Cross Section AM-AM (1:100)
Detailed Drawing of Part AK (1:500)
Detailed Drawing of Part AJ (1:500)
Cross Section AF-AF (1:100)
Assembly Part
Joining from an RTML to the accelerator room
See Drawing AH-AH
Cross Section AI-AI (1:100)
Cross Section AG-AG (1:100)
Cross Section AN-AN (1:100)
Position of a waterproof sheet
Cross Section AO-AO (1:200)
Positron side of the ILC tunnel
KE9080
19-3-03194
High Energy Accelerator Research Organization (KEK)


Detailed Drawing of Part B (1:1000)
For Cross Sections N, O, P, Q, and AQ, see other drawings and the cross section of the detector hall.
Cross Section AP-AP (1:100)
Cross Section I-I (1:100)
Cross Section J-J (1:100)
Cross Section K-K (1:100)
Cross Section L-L (1:100)
Cross Section M-M (1:100)
Cross Section R-R (1:400)
Notes
Correction
Revised by
Date
Name
Date
Made by
Tanaka
12/17/2019
Checked by
Approved by
Damping ring of the ILC
KE9080
19-3-03217
Projection method
m
Unit
High Energy Accelerator Research Organization (KEK)
Scale
1/50000
A0



Cross Section N-N (1:300)
See Cross Section O-O
Cross Section O-O (1:400)
Detailed Drawing of Part B (1:1000)
Cross Section P-P (1:300)
Cross Section Q-Q (1:400)
Cross Section AQ-AQ (1:300)
Notes
Correction
Revised by
Date
Name
Made by
Tanaka
12/17/2019
Checked by
Approved by
Cross sections of the detector hall of the ILC
KE9080
19-3-03218
High Energy Accelerator Research Organization (KEK)
Projection method
Unit
m
Scale
1/5000
A0