JAEA-Research
2011-001

Horonobe Underground Research Laboratory Project
Synthesis of Phase I Investigation 2001-2005
Volume “Geological Disposal Research”

Tomoo FUJITA, Naoki TANIGUCHI, Hiroya MATSUI, Kenji TANAI
Keisuke MAEKAWA, Atsushi SAWADA, Hitoshi MAKINO
Hiroshi SASAMOTO, Hideki YOSHIKAWA, Masahiro SHIBATA
Keiichiro WAKASUGI, Katsuhiro HAMA, Hiroshi KURIKAMI
Takanori KUNIMARU, Eiichi ISHII, Ryuji TAKEUCHI, Kunio OTA
Kaname MIYAHARA, Morimasa NAITO and Mikazu YUI

Geological Isolation Research Unit
Geological Isolation Research and Development Directorate

March 2011
Japan Atomic Energy Agency
日本原子力研究開発機構

JAEA-Research

本レポートは独立行政法人日本原子力研究開発機構が不定期に発行する成果報告書です。
本レポートの入手並びに著作権利用に関するお問い合わせは、下記あてにお問い合わせ下さい。
なお、本レポートの全文は日本原子力研究開発機構ホームページ（http://www.jaea.go.jp）
より発信されています。

独立行政法人日本原子力研究開発機構　研究技術情報部　研究技術情報課
〒319-1195　茨城県那珂郡東海村白方白根 2 番地 4
電話 029-282-6387, Fax 029-282-5920, E-mail:ird-support@jaea.go.jp

This report is issued irregularly by Japan Atomic Energy Agency
Inquiries about availability and/or copyright of this report should be addressed to
Intellectual Resources Section, Intellectual Resources Department,
Japan Atomic Energy Agency
2-4 Shirakata Shirane, Tokai-mura, Naka-gun, Ibaraki-ken 319-1195 Japan
Tel +81-29-282-6387, Fax +81-29-282-5920, E-mail:ird-support@jaea.go.jp

Horonobe Underground Research Laboratory Project
Synthesis of Phase I Investigation 2001-2005
Volume "Geological Disposal Research"

Tomoo FUJITA, Naoki TANIGUCHI, Hiroya MATSUI[+4], Kenji TANAI[+1], Keisuke MAEKAWA
Atsushi SAWADA, Hitoshi MAKINO[+2], Hiroshi SASAMOTO, Hideki YOSHIKAWA,
Masahiro SHIBATA[+3], Keiichiro WAKASUGI[*1], Katsuhiro HAMA[+4], Hiroshi KURIKAMI[+1],
Takanori KUNIMARU[+4], Eiichi ISHII[*2], Ryuji TAKEUCHI[+4], Kunio OTA[+2],
Kaname MIYAHARA, Morimasa NAITO[+2] and Mikazu YUI[+3]

Geological Isolation Research Unit
Geological Isolation Research and Development Directorate
Japan Atomic Energy Agency
Tokai-mura, Naka-gun, Ibaraki-ken

(Received January 12, 2011)

This report summarizes the progress of research and development on geological disposal during the surface-based investigation phase (2001-2005) in the Horonobe Underground Research Laboratory project, of which aims are to apply the design methods of geological disposal and mass transport analysis to actual geological conditions obtained from the project as an example of actual geological environment. For the first aim, the design methods for the geological disposal facility proposed in "H12 report (the second progress report)" was reviewed and then improved based on the recent knowledge. The applicability of design for engineered barrier system, backfill of disposal tunnel, underground facility was illustrated. For the second aim, the conceptual structure from site investigation and evaluation to mass transport analysis was developed as a work flow at first. Then following this work flow a series of procedures for mass transport analysis was applied to the actual geological conditions to illustrate the practical workability of the work flow and the applicability of this methodology. Consequently, based on the results, future subjects were derived.

Keywords : Geological Disposal, Design Methods, Solute Transport Analysis, Surface-based Investigation, Underground Research Laboratory

---

+1 Horonobe Underground Research Unit
+2 Research and Development Integration Unit
+3 Geological Isolation Research and Development Directorate
+4 Tono Geoscientific Research Unit
*1 On loan to Japan Nuclear Energy Safety Organization
*2 On loan to Nuclear Waste Management Organization

幌延深地層研究計画における地上からの調査研究段階
第 1 段階研究成果報告書
分冊「地層処分研究開発」

日本原子力研究開発機構

地層処分研究開発部門 地層処分基盤研究開発ユニット

藤田 朝雄，谷口 直樹，松井 裕哉[+4]，棚井 憲治[+1]， 前川 恵輔
澤田 淳，牧野 仁史[+2]，笹本 広，吉川 英樹，柴田 雅博，
若杉 圭一郎[*1]，濱 克宏[+4]，操上 広志[+1]，國丸 貴紀[+4]，石井 英一[*2]，
竹内 竜史[+4]，太田 久仁雄[+2]，宮原 要，内藤 守正[+2]，油井 三和[+3]，

（2011 年 1 月 12 日受理）

本報告書では，堆積岩で塩水系地下水を対象とした幌延深地層研究計画において段階的に得られる地質環境条件を一つの適用例として，第 1 段階である地上からの調査で得られた情報をもとに処分場の設計技術や性能評価技術それぞれの適用性について論じるとともに，必要に応じて実施した技術の改良や代替技術の開発状況を取りまとめた。

処分技術の信頼性向上では，最新の知見を踏まえ，第 2 次取りまとめにおいて示された処分場全体設計フローの更新や人工バリアなどの設計手法の詳細化，ならびに設計における地質環境条件の一般的な留意点や設計入力データ項目について整理を行なった。また，これらを踏まえ，幌延の地質環境条件を一例とした場合の施設設計，人工バリア設計及び閉鎖設計を通じて第 2 次取りまとめで採用された設計手法が適用可能であることがわかった。

安全評価手法の高度化については，第 2 次取りまとめにおいて示された安全評価手法を実際の地質環境に適用するために必要な具体的な作業をフローとして構築した。これに基づき，幌延の地質環境条件を一例として物質移行解析を行い，これらの検討を通じて第 2 次取りまとめの手法を堆積岩地域に適用した場合の調査から解析・評価にいたる一連の方法論及び，その過程で得られるノウハウや知見，調査や解析上の留意点を整理した。

核燃料サイクル工学研究所（駐在）：〒319-1194 茨城県那珂郡東海村村松 4-33
+1 幌延深地層研究ユニット
+2 研究開発統括ユニット
+3 地層処分研究開発部門
+4 東濃地科学研究ユニット
*1 原子力安全基盤機構へ出向中
*2 原子力発電環境整備機構へ出向中

Contents

目　　次

List of Tables

List of Figures

1. Introduction

The Horonobe Underground Research Laboratory (URL) Project is a comprehensive research and development (R&D) project aimed at studying the sedimentary formations in the region of Horonobe Town in Hokkaido, northern Japan. The URL project, which is managed by the Japan Atomic Energy Agency (JAEA; formerly the Japan Nuclear Cycle Development Institute – JNC), is one of two deep URL projects prescribed in “Long-Term Program on Research, Development and Utilization of Nuclear Energy” (Long-Term Program) (Atomic Energy Commission of Japan, 2000) and “Framework for Nuclear Energy Policy” (Framework) (Atomic Energy Commission of Japan, 2005), both of which were issued by the Atomic Energy Commission of Japan (AEC). The URL project consists of two research areas: ‘geoscientific research’ and ‘research and development on geological disposal technologies’. This report describes the geoscientific research, synthesising the results of investigations carried out during the surface-based investigation phase – Phase I of the Horonobe URL project.

Section 1.1 provides an overview of the URL project within the context of the Japanese geological disposal programme; Section 1.2 describes the R&D programme of the project and Section 1.3 outlines the strategy for synthesising the results of the Phase I investigations.

1.1 General background

1.1.1 URL projects

Building on the technical basis provided by JNC’s report “H12: Project to Establish the Scientific and Technical Basis for HLW Disposal in Japan” (H12 report) (JNC, 2000), the geological disposal programme for high-level radioactive waste (HLW) in Japan has moved from the stage where only R&D is performed to the implementation stage, in which a geological disposal project and safety regulations will be implemented. Necessary R&D activities will be continued in parallel. In June 2000, the Specified Radioactive Waste Final Disposal Act (the Act) was promulgated and, based on this legislation, the Nuclear Waste Management Organization of Japan (NUMO) was established as the implementation body for the geological disposal of HLW in October of the same year. The Nuclear Safety Commission of Japan (NSC) issued the “Basic Concept of Safety Regulation on HLW Disposal” First Report (Nuclear Safety Commission of Japan, 2000). Thus, the preliminary framework for the geological disposal project and the associated safety regulations has been established.

AEC then specified a new national programme framework in its Long-Term Program (Atomic Energy Commission of Japan, 2000), which outlined the activities leading up to final disposal of HLW and who would be responsible for performing these. NUMO “*...should take charge of developing those consistent with the safe implementation of the final disposal project and with* *the improvement of its economic performance and efficiently"*. The government and related organisations *"...should actively push forward with research and development projects necessary for safety regulation and safety assessment of the final disposal, with fundamental research and development activities, including scientific studies of the deep geological environment, and with development of technologies to enhance the reliability of geological disposal technology"*. In particular, JNC was assigned responsibility to *"...steadily carry on research and development activities to verify the reliability of geological disposal technologies and to establish a safety assessment method, using research facilities for deep geological environments and the Quantitative Assessment Radionuclide Migration Experiment Facility in Tokai village"*.

In accordance with the responsibilities assigned to it in the Long-Term Program (Atomic Energy Commission of Japan, 2000), JNC formulated an R&D programme for the stages following submission of the H12 report. This was published as the "Generic Programme for R&D on Geological Disposal of HLW" (Generic Programme) (Evaluation Committee of Research and Development Activities, 2001). In the Generic Programme and within the context of improving 'the technical reliability of geological disposal in Japan', demonstrated in H12 report, and enhancing the technical basis supporting the disposal project and the associated safety regulations, two goals were set for the R&D programme: 'confirmation of the applicability of disposal technologies to specific geological environment' and 'understanding of the long-term behaviour of the geological disposal system' (Figure 1.1.1-1). The aim of the former was to confirm the reliability and practicability of technologies developed to date by applying them to specific geological environments; the latter was intended to improve the reliability of the evaluation by increasing the understanding of various phenomena that affect the disposal system and to improve the associated models and databases. The URLs were focal points for achieving the first goal. In the Generic Programme, the R&D programme was divided into two areas: 'R&D on geological disposal technology' and 'geoscientific research'. The former involves improving the reliability of the disposal technologies and developing advanced safety assessment methodologies, while the latter forms the necessary basis for the former. Specific R&D items were set for each area. Since JNC was merged with the Japan Atomic Energy Research Institute (JAERI) to form JAEA, R&D has continued in line with the above two R&D goals. A new fundamental policy report of JAEA entitled "Plan for Meeting the Midterm Goal – Midterm Plan (October 1, 2005 to March 31, 2010)" (Midterm Plan) (Japan Atomic Energy Agency, 2005) states that JAEA will pursue R&D in the fields of geoscientific research and geological disposal technologies. The URL projects at Horonobe in Hokkaido, northern Japan and Mizunami in Gifu, central Japan are mentioned specifically in the Midterm Plan.

The R&D activities summarised in the H12 report were carried out to demonstrate the basic technical feasibility of geological disposal in Japan, without specifying any particular geological environment. In order to improve the technical reliability of geological disposal and enhance the technical basis for the disposal project, it was important that the practicability and reliability of the technologies used in the H12 report should be demonstrated by applying them to specific geological environments; URLs can be used for this purpose.

It was proposed that "*two or more URLs should be constructed, considering the range of characteristics and distribution of the geology of Japan*" *(Atomic Energy Commission of Japan, 1994)*. JNC therefore set up the Horonobe URL project (JNC, 2001)[8] for investigating sedimentary rock with saline groundwater and the Mizunami URL project (JNC, 2002)[9] for crystalline rock with fresh groundwater (Figure 1.1.1-2). Both projects consist of three phases that will extend over a period of around 20 years (Nuclear Waste Management Organization of Japan, 2001): surface-based investigations (Phase I), investigations during tunnel excavation (Phase II) and investigations in the underground facilities (Phase III). It should be noted that the investigations for repository site selection, which will be carried out by NUMO, will also proceed stepwise, with literature surveys, preliminary investigations and detailed investigations (surface-based investigations in the early stages and those in underground facilities in the later stages) over a period of around 20 years, as specified in the Act. Phase I of the Horonobe URL project corresponds to the preliminary investigations and the first half of the detailed investigations in the NUMO programme and Phases II and III to the detailed investigations using underground facilities. The technical basis for the preliminary investigations has been developed by synthesising the results in the H12 report and investigations carried out during Phase I of the URL projects. It is of importance that the reliability of surface-based investigations and associated modelling techniques should be improved during Phase II investigations; for instance, geological environment models developed during Phase I should be tested using geological data obtained during Phase II. This is defined also in the national "R&D Programme for the Geological Disposal of HLW" (Agency for Natural Resources and Energy and Jpan Atomic Energy Agency, 2006); the goal of the fundamental R&D in Phase 2 (2006–2010) will be to establish the technical basis for surface-based investigations and that in Phase 3 (after 2011) to establish the technical basis for investigations using underground facilities.

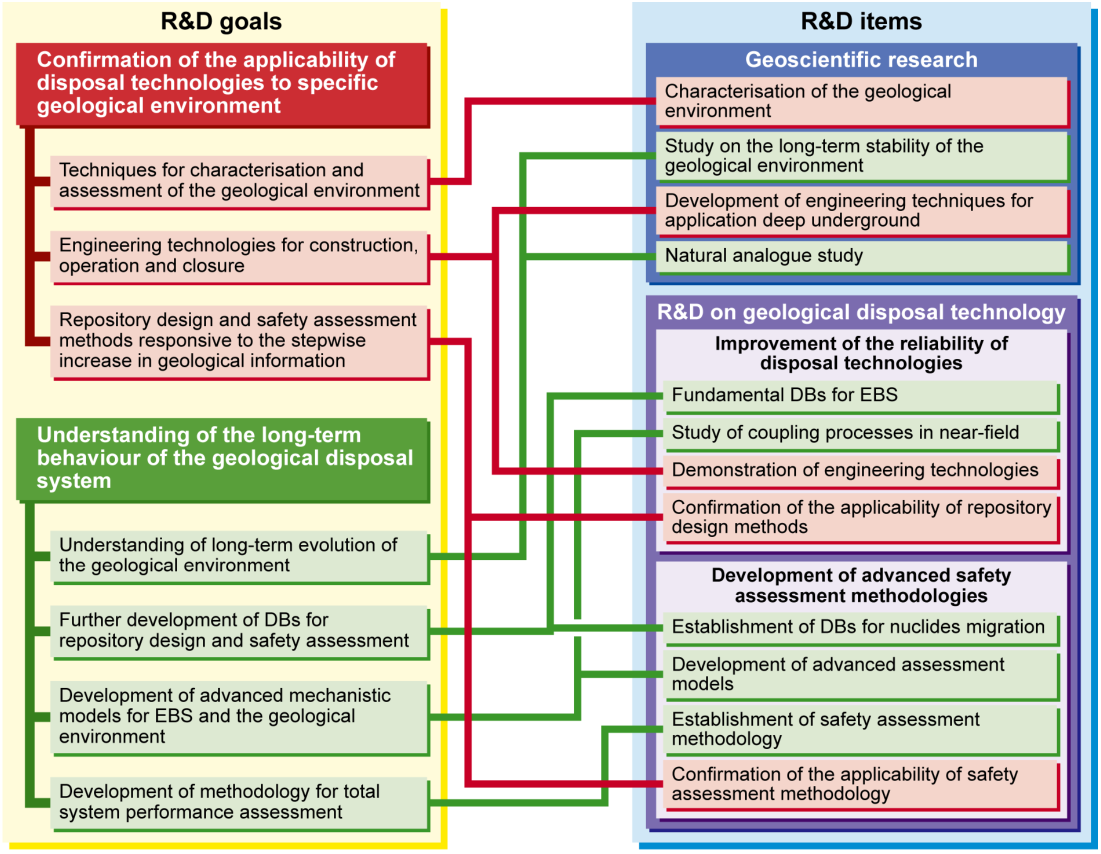


Figure 1.1.1-1 Goals of the R&D plan (2001–2005) mapped onto specific work areas

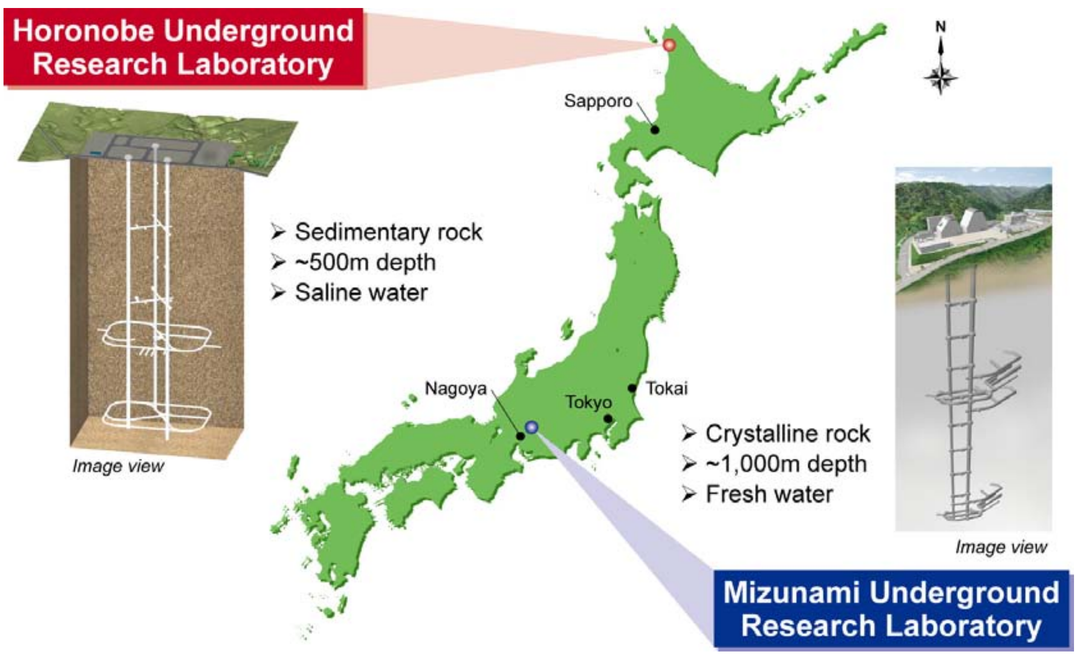


Figure 1.1.1-2 Two URL projects in Japan

1.1.2 R&D goals of the URL projects

The primary goal of the R&D in the URL project is to improve the reliability of geological disposal technologies by applying them to specific geological environments. In the H12 report, techniques and methods for characterising the geological environment were developed based on studies carried out in existing tunnels in the Tono mine (sedimentary rock) and the Kamaishi mine (granitic rock), as well as joint research projects in overseas URLs. After comparing and reviewing the information acquired in these studies and from key literature, it was demonstrated that a safe repository could be designed and constructed economically based on current engineering technologies or if necessary future developments thereof and that the long-term safety of geological disposal could be assessed by current modelling and simulation techniques. It can be concluded that the H12 report successfully demonstrated the feasibility of geological disposal in principle combining information from various fields.

The reliability of geological disposal technologies will be evaluated based on information on the geological environment accumulated with the progress of the URL project, with investigations advancing stepwise starting from undisturbed conditions. A stepwise approach helps in enhancing and ensuring the depth of understanding of the deep geological environment and eventually developing a sound technical basis for systematically characterising the deep geological environment. Design and construction of URLs based on the geological information acquired will result in developing engineering technologies relevant to geological disposal programmes. A key technical challenge for the geoscientific research in the URL project will thus be to establish the basis for techniques for characterising the geological environment and engineering technologies for use in the deep subsurface. In addition, in the Horonobe URL project, techniques for systematically investigating historical records of natural phenomena, such as seismic activity, uplift, erosion etc., and acquiring information on the evolution of the geological environment up to the present will be developed as a case study.

The information and knowledge acquired through geoscientific research will be used as key input for R&D aimed at improving the reliability of repository engineering technologies and developing advanced safety assessment methodologies. In addition, rock and groundwater samples obtained in the geoscientific research will be utilised in the R&D. At the same time as enhancing confidence in the reliability of the underlying engineering technologies for an engineered barrier system (EBS) and the applicability of design and safety assessment methodologies, as shown in the H12 report, it will be important to iterate such developments based on the increasingly detailed geological information accumulated. In this way, the applicability of geological disposal technologies can be confirmed. Consequently the relationship between the degree of understanding of the geological environment and the sensitivity of the characteristics of the geological environment in a safety assessment or the required volume of investigations can be reviewed by feeding the confirmed results back into geoscientific research. Key issues for the planning and/or evaluation of repository design and construction can be identified and compiled as a knowledge base, depending on the progress of the stepwise investigations and the depth of understanding of the geological environment. Laboratory and engineering-scale experiments under controlled conditions or experiments using radionuclides also provide an understanding of various phenomena relevant to the geological disposal system. In the Horonobe URL project, R&D on geological disposal technology is conducted in parallel with geoscientific research, which, particularly in the phases subsequent to Phase II of the project, will involve using drifts for experiments to confirm EBS construction technologies and their performance in sedimentary rock and in situ experiments for improving the reliability of safety assessment models (Evaluation Committee of Research and Development Activities, 2005).

1.1.3 Role of the URL projects in Japan

The role of the URL projects in Japan is to develop and confirm the relevant technologies that will be applied in a real geological environment. This is achieved by maintaining an awareness of the overall goal – accurate understanding of the geological environment – while setting tasks such as data acquisition and evaluation of the applicability of the technologies. This will lead to optimisation and improvement of the technologies, as well as an understanding of the adequacy and limitation of technologies. Understanding the limitations of the technologies will form a basis for the detailed planning of repository implementation and the formulation of realistic safety regulations. Since the waste disposal project will proceed stepwise, as specified in the Act, and safety regulations will be developed in accordance with the progress of the waste disposal project, these activities should be preceded by R&D that will provide a sound technical basis for them. It is of particular importance that the results from each URL investigation phase should be synthesised and reported with an appropriate lead-time for the site selection process. This is also essential for reviewing the design and safety assessment methodologies for the geological disposal system using information on the specific geological environments, which will be acquired through the URL programme. This will thus ensure adequate techniques for characterising the geological environment efficiently and an efficient approach to reviewing the design and safety assessment methodologies for the geological disposal system based on limited stepwise information on the geological environment.

When designing a disposal system and evaluating its safety based on the results of characterisation of the geological environment, major challenges include understanding the spatial heterogeneities of the actual geological environment and addressing the uncertainties associated with these heterogeneities. This involves frequent trial and error and repeated feedback. In general, the depth of understanding will increase in proportion to the extent of investigations. However, the further the investigations progress, the less the increase in understanding and cost effectiveness of the efforts become. Detailed investigations of the geological environment generally involve high costs and no investigations, however detailed, will result in complete understanding. The geological disposal system is inherently associated with various degrees of uncertainty and the degree of conservativeness to be taken into consideration in the repository design will vary depending on the level of uncertainty associated with the understanding of the geological environment. For the actual disposal project, this will be reflected in the degree of conservativeness in the repository design and, subsequently, in the construction costs. Understanding the geological environment and reflecting the remaining uncertainties in the margins included in the design and safety assessment is thus a complex issue and the investigations have to reduce the uncertainties to an acceptable level. Hence, in each investigation in the stepwise site selection process, it will be important to examine the objectives and content of the investigations by assessing the level of understanding to be achieved and the requirements to be considered, as well as identifying issues and uncertainties to be carried over to the next stage. Even for investigations of a similar scale, the accuracy of the results and the depth of understanding obtained will vary depending on the geological and geomorphic conditions and the social environment at the site. In order to reflect such realistic conditions, it is important not only to prepare catalogues of investigation techniques and equipment, but also to accumulate experience at the site and to learn from actual case studies. The experience and knowledge gained in the URL projects should support methodologies for investigations for repository site selection and be used to establish safety regulations. For the URL projects, efforts should be made to build a knowledge base and to systematise multidisciplinary results and technological successes as well as experience with failures, recognising that the latter are also important aspects of the ongoing R&D. The approach to the investigations in the URL projects is thus not linear, but involves a process in which planning, execution and evaluation are iterated in each phase. Recognising the relationship between the progress of investigations and the depth of understanding and reduction of uncertainties will result in optimisation of the investigation programme as a whole, with the findings of preceding phases being reflected in subsequent phases.

It is stated in the Long-Term Program (Atomic Energy Commission of Japan, 2000) that *"The research facility for deep geological environment will serve not only as a place for scientific investigation, but also as a place for deepening public understanding of research and development activities related to the geological disposal of waste. Accordingly, this research facility project should be clearly distinguished from the disposal facility"*. It is worth noting in this respect that an agreement between JAEA and the local government restricts JAEA's activities in the sense that *"JAEA will never bring or use HLW in the URLs and JAEA will never lend or transfer the URL facilities to the implementing entity"*. Collaborative research with domestic and overseas organisations and experts will also be pursued intensively in the URL projects. The public will have the opportunity, in the URLs, to experience the deep geological environment first-hand and to increase their awareness of geological disposal and the associated R&D activities. The investigation programme and results of the URL projects will be open to the local and national public.

1.2 The Horonobe URL project

1.2.1 R&D goals and items

As described in the Generic Programme (Figure 1.1.1-1), the Horonobe URL project involves geoscientific research and R&D on geological disposal technology for sedimentary formations, with three major goals:

- to establish the basis for techniques for characterising the deep geological environment;
- to develop the basis for engineering technologies for use in the deep underground;
- to confirm the applicability of geological disposal technologies in specific geological environment.

With regard to geoscientific research, the R&D item 'characterisation of the geological environment' prescribed in the Generic Programme can be divided into two sub-items: 'development of techniques for investigating the geological environment' and 'development of techniques for long-term monitoring of the geological environment'. Coupled with 'studies on the long-term stability of the geological environment' and 'development of engineering techniques for application deep underground', this constitutes four R&D items. R&D on improving the reliability of disposal technologies consists of three areas: 'study of coupling processes in near-field', 'demonstration of engineering technologies' and 'confirmation of the applicability of repository design methods'. Development of advanced safety assessment methodologies consists of two areas, viz. 'development of advanced assessment models' and 'confirmation of the applicability of safety assessment methodology'. These areas will be addressed in three, partially overlapping, phases extending over around 20 years from the period before construction of the URL up to the phase after completion of construction of the underground facilities (Figure 1.2.1-1). The Phase I surfaced-based investigations will cover seven R&D items, other than 'study of coupling processes in near-field' and 'development of advanced assessment models'. The Phase II investigations during construction of the underground facilities will involve eight R&D items and Phase III will address all nine R&D items.

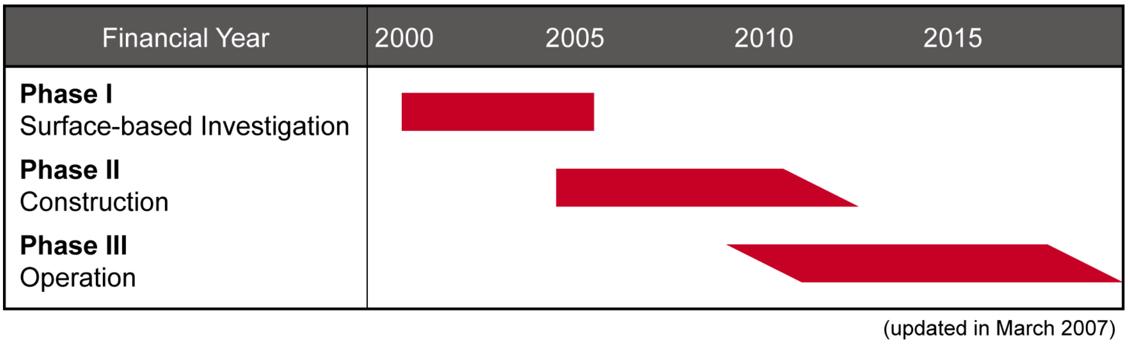


Figure 1.2.1-1 Timetable of the Horonobe URL project

The outline geological setup of the region of Horonobe Town consists of Palaeogene to Early Pleistocene sedimentary sequences, terrace sediments after the Middle Pleistocene and Holocene sediments (sand dunes and alluvium), which are underlain by a Cretaceous basement (Oka and Igarashi, 1997). Two large-scale faults and a fold structure with an NNW strike have been identified (Oka, 1986; Akiyama and Hoyanagi, 1990) and active Quaternary structures such as faults and flexures have also been identified to the east and west of the town (Nakata and Imaizumi, 2002). It is also known that the Neogene sedimentary sequences to be characterised (the Wakkanai and Koetoi Formations) have generally low permeability and contain two types of groundwater, saline and fresh, which dissolve gases.

1.2.2 Progress to date

Under an agreement between the Horonobe Town, Hokkaido Prefecture and JNC (now JAEA), with the participation of the Science and Technology Agency (now the Ministry of Education, Culture, Sports, Science and Technology), dated November 2000 on ‘Geoscientific Research in Horonobe’, Phase I of the Horonobe URL project was initiated in March 2001. The agreement specified that JNC would never bring or use HLW in the area covered by the project and would never lend or transfer the URL facilities to the implementing entity. JNC was also obliged to close the surface facilities and backfill the underground facilities after completion of the project. The area is also excluded from being a site for a radioactive waste repository or an interim storage facility for radioactive waste. In July 2002, based on surveys of existing information and aerial and ground reconnaissance surveys on a regional scale in the previous year and taking into consideration preliminary requirements on the geological environment and safety, as well as social and environmental constraints, an area of 3 km × 3 km square in the Hokushin district in the north-central part of Horonobe Town was selected as the main area for the surface-based investigations (URL area). A site for constructing the underground and surface facilities (URL site) was subsequently selected in the URL area, 3 km from central Horonobe. The selection was based on social conditions and the availability of infrastructure, including roads and land use restrictions, in addition to available geological and hydrogeological information. JNC purchased the site in March 2003 (Figure 1.2.2-1). In June 2003, preparation of the land started and, in April 2005, construction of the underground facilities was initiated, as were the Phase II investigations (Matsui et al., 2006). Excavation of the Ventilation Shaft began in November 2005. Phase I investigations continued for approximately five years up to the end of March 2006, during which time aerial geophysical surveys and various surface-based investigations were conducted and supporting laboratory investigations using field information and rock and groundwater samples taken.

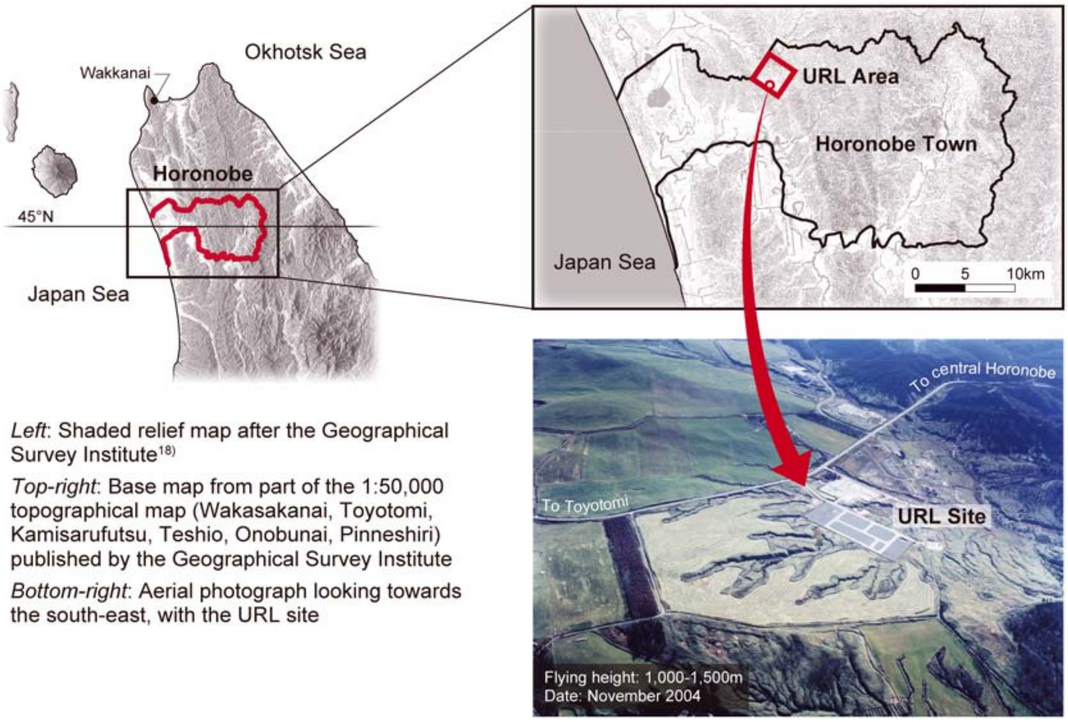


Figure 1.2.2-1 Location of the URL area and URL site in Horonobe Town, northern Hokkaido

1.3 Synthesis of the Phase I investigations

1.3.1 Aims and scope

Synthesis of the results from the Phase I investigations involved systematically compiling the results and knowledge from surface-based investigations up to the end of March 2006. This synthesis is important for evaluating the Phase I goals for which the investigations have been carried and for identifying issues to be addressed in the future, as well as providing guidelines for the investigations in Phase II and subsequent phases. The results of the synthesis should be used as a knowledge base that supports both NUMO's repository implementation (e.g.

preliminary investigations and detailed surface-based investigations for site selection) and the formulation of safety regulations by national agencies (e.g. establishing siting factors for the selection of detailed investigation areas and guidelines for the safety review). The results of the Phase I investigations were incorporated into supporting reports 1–3 of the project “H17: Development and Management of the Technical Knowledge Base for the Geological Disposal of HLW” (H17 report) (JNC, 2005a; 2005b; 2005c) published in September 2005.

The strategy for the synthesis of Phase I was specified as compiling the results of the investigations and associated knowledge in accordance with the goals of Phase I and defining the investigation needs in Phase II and subsequent phases. More specifically, all the results obtained during Phase I, including those obtained after publication of the H17 report, will be compiled and synthesised and, based on future issues identified in the H17 report and comments provided on the H17 report, the techniques for characterising the geological environment and engineering technologies for deep underground application, developed during Phase I, will be reviewed. Also, from the viewpoint of establishing a knowledge base for geological disposal technologies, it is important to document the full spectrum of experience, including both successes and failures.

The report on the Phase I synthesis of the Horonobe URL project consists of two volumes: “Geoscientific Research” and “Geological Disposal Research”. The present report, Volume “Geoscientific Research”, documents the results of the following R&D items: ‘development of techniques for investigating the geological environment’, ‘development of techniques for long-term monitoring of the geological environment’, ‘studies on the long-term stability of geological environment’ and ‘development of engineering techniques for application deep underground’, while the report, Volume “Geological Disposal Research”, presents the results of ‘demonstration of engineering technologies’, ‘confirmation of the applicability of repository design methods’ and ‘confirmation of applicability of safety assessment methodology’.

1.3.2 Structure of the report on geological disposal technologies

In light of stepwise site selection approach prescribed in the Act, it is important to apply the geological disposal technologies established in the H12 report to relevant geological environments and to compiling the resulting experience and knowledge. It is also important to improve the technologies and to develop alternatives in order to solve any difficulties encountered during application and to present the results as a technical basis for the disposal project and the safety regulations.

Given this perspective, the report discusses the applicability of design and safety assessment technologies for the repository based on information obtained from surface-based investigations in Phase I and summarizes the improvement of technologies and development of alternative technologies as required, using the geological conditions obtained during the Horonobe URL project for sedimentary rock with saline groundwater as a case study. It should be noted that, since the project is not for an actual repository, the technologies addressed in this report concern only those for design and safety assessment of some components of the geological system and not for the total system; state-of-the art scientific knowledge has been incorporated into the discussion.

This report consists of four chapters. Chapter 2 is on improving the reliability of disposal technologies and demonstration of the applicability of the repository design includes a review of design flow for the repository shown in the H12 report, refinement of design methods for the engineered barriers, important aspects related to geological environment conditions and input parameters for the design. Based on this generic discussion, it also addresses the applicability of the design method used in the H12 report through setting depth and input parameters for the design, implementation of the design of the facilities and the engineered barriers and closure of the repository using the aforementioned parameters for the geological environment data in Horonobe (depth and design input parameters) as a case study. It concludes with open and future issues identified during the course of the discussions in the report. Chapter 3 describes specific approaches for safety assessment according to the work flow outlined for applying the safety assessment methods shown in the H12 report to relevant geological environments. Mass transport analyses and their results based on the work flow for the geological environment characterized by the surface-based investigations in Horonobe were also described. It also describes methods outlined in the H12 Report for investigation and analysis of sedimentary rock formations and the know-how and points to be kept in mind during investigation and analysis obtained through these discussions. Finally Chapter 4 summarizes the results of the research on geological disposal technologies in Phase I and discusses future perspectives, including programs for Phase II and subsequent phases.

References

Atomic Energy Commission of Japan: "Long-Term Program on Research, Development and Utilization of Nuclear Energy", 24th Nov. 2000 (2000). (in Japanese)

Atomic Energy Commission of Japan: "Framework for Nuclear Energy Policy", 11th Oct. 2005 (2005). (in Japanese)

Japan Nuclear Cycle Development Institute (JNC): "H12: Project to Establish the Scientific and Technical Basis for HLW Disposal in Japan", Project Overview Report, JNC TN1410 2000-001 (2000).

Nuclear Safety Commission of Japan: "Basic Concept of Safety Regulation on High-Level Radioactive Waste Disposal", First Report, Special Committee on Safety Regulation of Radioactive Waste, 6th Nov. 2000 (2000). (in Japanese)

Evaluation Committee of Research and Development Activities, Japan Nuclear Cycle Development Institute: "Assessment Report of Research and Development Activities in FY2001 – Generic Programme for Research and Development on Geological Disposal of High-Level Radioactive Waste", Interim Report, JNC TN1440 2001-008 (2001). (in Japanese)

Japan Atomic Energy Agency: "Plan for Meeting the Midterm Goal – Midterm Plan (October 1, 2005 to March 31, 2010)", 3rd Nov. 2005 (2005). (in Japanese)

Atomic Energy Commission of Japan: "Long-Term Program on Research, Development and Utilization of Nuclear Energy", 24th Jun. 1994 (1994). (in Japanese)

Japan Nuclear Cycle Development Institute (JNC): "Deep Geological Research Laboratory (Tentative Name) Project – Plan of Surface-Based Investigation (Phase I)", JNC TN1410 2001-001 (2001). (in Japanese)

Japan Nuclear Cycle Development Institute (JNC): "Master Plan of the Mizunami Underground Research Laboratory Project", JNC TN7410 2003-001 (2002).

Nuclear Waste Management Organization of Japan: "Basic Strategy for Selection Procedures for Preliminary Investigation Areas for the Disposal of HLW" Official Gazette, Extra No. 238, 8th Nov. 2001 (2001). (in Japanese)

Agency for Natural Resources and Energy and Japan Atomic Energy Agency: "Research and Development Programme for the Geological Disposal of HLW", Dec. 2006 (2006). (in Japanese)

Evaluation Committee of Research and Development Activities, Japan Nuclear Cycle Development Institute: "Assessment Report of Research and Development Activities in FY2004 – Horonobe Underground Research Laboratory Project", Interim Report, JNC TN1440 2005-002 (2005). (in Japanese)

Oka, T. and Igarashi, Y.: "Latest Cenozoic formations in the northern part of Teshio Plain, Hokkaido –

Sedimentary facies and pollen stratigraphy", In: M. Kawamura, T. Oka and T. Kondo (Eds.) Commem. Vol. Prof. M. Kato, pp. 341-365 (1997). (in Japanese with English abstract)

Oka, T.: "Distribution and tectonic evolution of Late Cenozoic basins in Hokkaido", Monogr. Assoc. Geol. Collab. Jpn., No. 31, pp. 295-320 (1986). (in Japanese with English abstract)

Akiyama, M. and Hoyanagi, K.: "Neogene, Tenpoku Area", Regional geology of Japan part 1: Hokkaido, Kyoritsu Shuppan, pp. 105-106 (1990). (in Japanese)

Nakata, T. and Imaizumi, T. (Eds.): "Digital Active Fault Map of Japan", Univ. Tokyo Press (2002). (in Japanese)

Matsui, H., Niizato, T. and Yamaguchi, T. (Eds.): "Horonobe Underground Research Laboratory Project Investigation Report for the 2005 Fiscal Year", JAEA-Research 2006-073 (2006). (in Japanese with English abstract)

Japan Nuclear Cycle Development Institute (JNC): "H17: Development and Management of the Technical Knowledge Base for the Geological Disposal of HLW", Supporting Report 1: Geoscience study, JNC TN1400 2005-014 (2005a). (in Japanese)

Japan Nuclear Cycle Development Institute (JNC): "H17: Development and Management of the Technical Knowledge Base for the Geological Disposal of HLW", Supporting Report 2: Repository engineering technology, JNC TN1400 2005-015 (2005b). (in Japanese)

Japan Nuclear Cycle Development Institute (JNC): "H17: Development and Management of the Technical Knowledge Base for the Geological Disposal of HLW" Supporting Report 3: Safety assessment method, JNC TN1400 2005-016 (2005c). (in Japanese)

2. Improving the reliability of disposal technology and demonstrating the applicability of repository design

To ensure the long-term safety of geological disposal, it is necessary to optimize repository design in accordance with the characteristics of potential sites, which will be clarified in increasing detail with the siting process consisting of literature surveys, preliminary investigations and detailed investigations (NUMO, 2004a).

In this stepwise siting process, the disposal concept will be developed iteratively and will include (a) the design and layout of the surface and underground facilities, including the engineered barrier, system (EBS), (b) the construction, operation, closure and monitoring of the repository, (c) safety during operations and long-term safety after closure and (d) environmental and socio-economic impacts. The concept will also involve consideration not only of long-term safety, but also of aspects such as economy and efficiency that are important for the actual construction work (NUMO, 2004a).

The surface-based investigations conducted as part of the Horonobe URL project are comparable with those to be carried out in the preliminary investigation phase and in Phase I of the detailed investigations for the actual disposal project.

The preliminary investigations provide only generic information on the spatial distribution of the features of the geological environment. Designing the repository and the engineered barriers based on these investigations alone would be insufficient for demonstrating the safety of the disposal system in detail, but they would provide the information needed for comparing several candidate sites in order to identify the sites at which the next step (i.e. the detailed investigations) should take place.

The repository and engineered barrier system design that will form part of Phase I of the detailed investigations will use more accurate and reliable information on the geological environment than that available at the preliminary investigation stage.

Recognizing the above, and in line with the concept presented in the H12 Report, this chapter describes a design methodology based on information on the geological environment that would obtained in Phase I of the detailed investigations in the disposal project. This study was made under the following assumptions:

- A vertical emplacement disposal system
- Disposal tunnels and disposal pits with the cross-section shown in Figure 2-1

The repository layout and the technologies for construction, operation and closure are not included in this study for the following reasons.

The repository layout involves the location of the repository, the geometry, size and layout of the disposal panels and the layout of the access, main and connecting tunnels. Some requirements to be covered in the design have already been presented. One of these is that the facility should be designed to have no significant adverse effects on the nuclide migration retardation capacity, which is an important factor in planning the layout, direction and number of the disposal panels and tunnels. To address this requirement, analysis of groundwater flow and mass transport and acquisition of information on a geological scale would be required. However, the purpose of the URL project is to perform fundamental research and development and it is not intended to demonstrate the validity of all the factors relating to the disposal project. Not all the information required to address these issues will be available and the repository layout is therefore not covered in this study.

In terms of studies on disposal technologies for repository construction, operation and closure JAEA has not been involved in the development of hardware related to repository operation and closure, in accordance with the role sharing agreement among relevant organizations. Chapter 5 of a separate volume on geoscientific research outlines studies on the construction technology for the underground facilities and this subject is also excluded from this report.

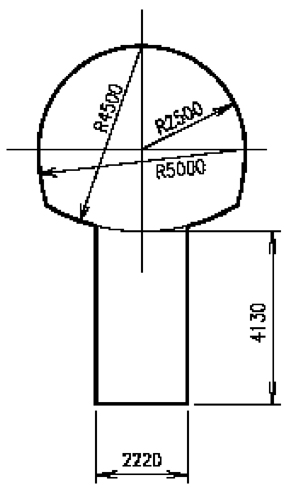


Figure 2-1 Cross-section of disposal tunnel and deposition hole（JNC，1999）

2.1 Basic design methodology and procedure

2.1.1 Study of repository design methods

The basic design flow used in the H12 Report (H12 design flow) is shown in Figure 2.1.1-1. According to this, the prerequisites for the design (such as disposal depth) are determined first; design requirements for both the surface and underground facilities are then specified, the design procedure is determined for each component of the facilities and a design analysis is conducted to define the specifications of these components (JNC, 1999). The design requirements should be defined considering all the required functions of the engineered barriers, the surface and the underground facilities and technologies that are already available or will be available in the near future, considering both operational and economic aspects, while assigning first priority to the long-term safety of the repository. The geological information on a proposed site, such as rock types and groundwater properties, also need to be considered. A new design methodology was developed for the overall repository system in this study, based on the concept and methods in H12 and incorporating the latest knowledge available in order to establish a systematic methodology using actual geological environment conditions with the perspective of modification of the H12 design flow.

More specifically, based on the premise that the methodology has to be transparent and traceable for third parties, interrelated items and influence factors among the individual designs were clarified (Table 2.1.1-1) in order to (1) demonstrate backfill and plug designs and (2) define design procedures taking the mechanical interaction between the buffer and the overpack into account

A study was made of how changes in buffer and backfill specifications will affect the specification of the required corrosion allowance for the overpack. Based on the result, the H12 design flow was revised as shown in Figure 2.1.1-2 (Matsui et al., 2005a).

Major revisions of the H12 design flow and the reasons for these are as follows:

(a) Evaluation of the mechanical stability of underground tunnels has to be carried out before the design of the engineered barriers because this is critical for determining the required dimensions of the concrete support for the disposal tunnels and judging the need for support for disposal pits, which are in turn required for determining the specifications of the backfill and buffer.

(b) The corrosion allowance for the overpack is determined as the sum of the depth of corrosion caused by reduction of water and that of corrosion due to oxidation. For the latter, the calculation is made assuming that all the oxygen contained in the buffer and backfill materials is consumed by the oxidation process. The design of the backfill has to be carried out before the overpack design.

(c) In order to develop the design procedure taking mechanical interactions between the buffer and the overpack into account, the design process was divided into two independent design of the buffer and confirmation of buffer specifications.

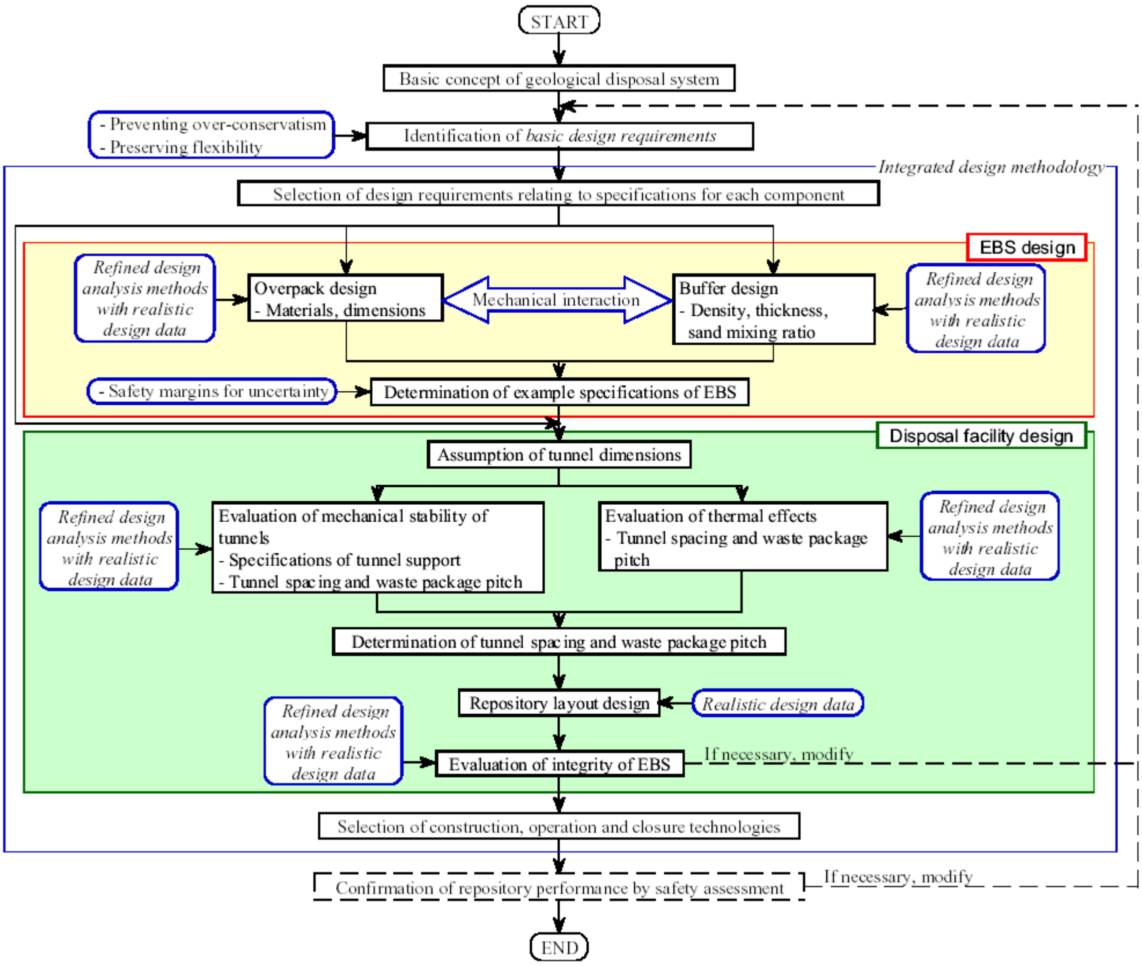


Figure 2.1.1-1 Design flow of disposal facility proposed in H12 report（JNC, 1999）

Table 2.1.1-1 Related other design items
and influence factors on vertical emplacement concept

	Design requirement	Design item	estimation item	factor		impact on other design item
overpack	corrosion resistance / corrosion allowance	corrosion allowance	corrosion depth	Corrosion due to oxygen, corrosion due to water	・oxigen in the buffer and buckfill ・adsorbed oxygen in bentonite	・density, sand mixing ratio and thickness of buffer ・density and sand mixing ratio of buckfill ・cross-section of tunnel, waste package pitch
	pressure resistance	thickness, shape, material	stress	Stress to occur in buffer	Swelling pressure andconsolidation stress of buffer	density, thickness,and sand mixing ratio of buffer
	radiation shielding	thickness	Localized corrosion	Cathodic current by radiolysis of groundwater	Radiation field, migration of oxidant	density, thickness,and sand mixing ratio of buffer
buffer material	stress buffering properties	thickness, specification of material	stress	Stress to occur in buffer	Corrosion expansion of overpack	Material, thickness and shape of overpack
	mechanical support of the overpack	Specification of material	Strength of buffer	Loading at lower part of buffer	Self-weight of overpack	Material and size of overpack
					Self-weight of buckfill	Density of buckfill, cross-section of tunnel
buckfill mnaterial	Countermeasure against extrusion of buffer into tunnels	Specification of material	Extrusion volume of buffer	Swelling and consolidation behaviour of buffer	Swelling property of buffer	specification of material of buffer
	To restrict lower than requirement level of permeability in the tunnel	Specification of material	self-sealing and hydraulic properties	Swelling and permeability of buckfill	Swelling and permeability of buckfill	Degree of deterioration, Thickness of tunnel support
disposal facility	Temperature limit of near-field	waste package pitch for horizontal emplacement	Thermal increase of buffer	Thermal diffusion	Thermal property of near-field	Specification of buffer and buckfill, cross-section of tunnel

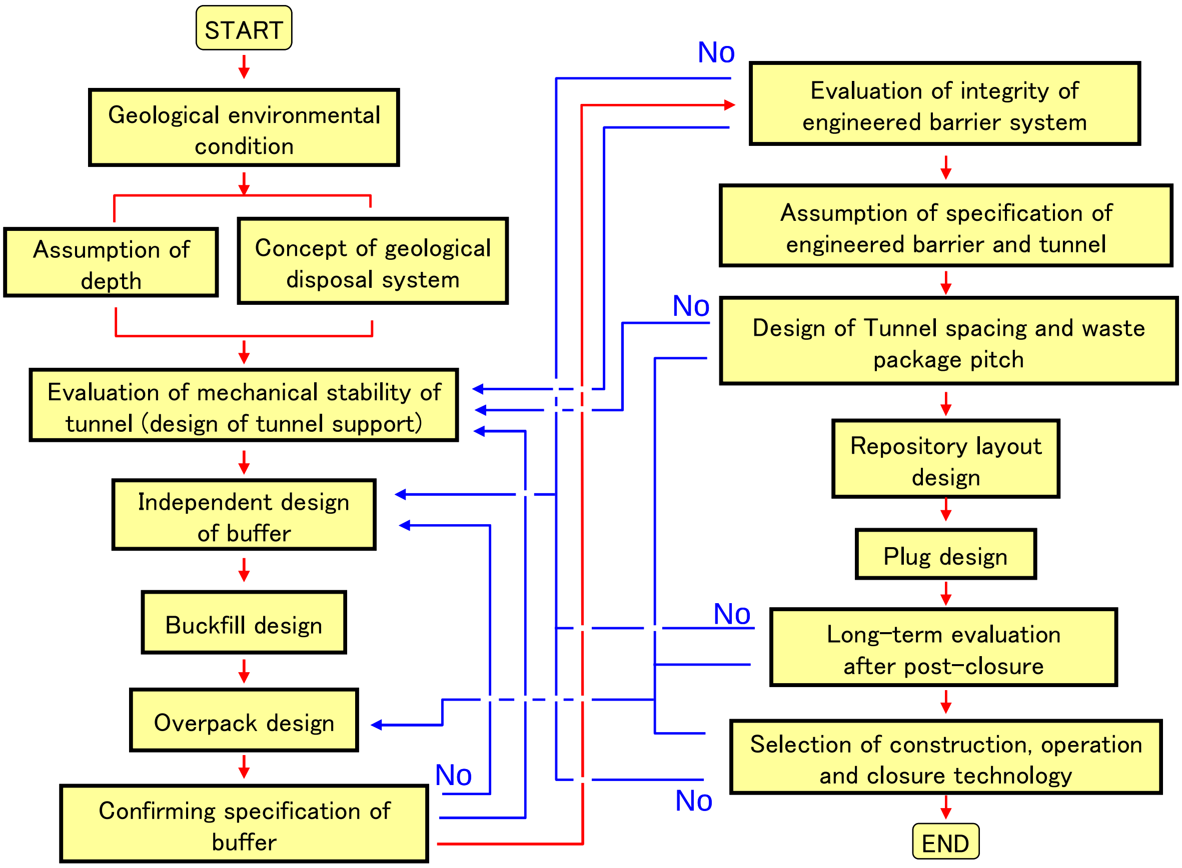


Figure 2.1.1-2 Improved design flow of geological disposal facility
(Revison of Matsui et al., 2005a)

2.1.2 Points to note concerning geological environment conditions on disposal design

The performance of the individual components specified in the design requirements might be affected by geological conditions at the specific site, including groundwater chemistry, pH, redox conditions and temperature, as well as other factors such as support materials and heat generation by the HLW glass. It is therefore important for the design after the repository site has been selected to define the design requirements and geological conditions that affect them, and to evaluate what type of influence the conditions would have on each design requirement. The relationship between geological conditions that might influence the design of the engineered barriers (overpack and buffer) and of the closure system (backfill and plugs), and the necessary measures to be taken in the design, were summarized as shown in Tables 2.1.2-1 to 2.1.2-4 (Kurihara et al., 2004; Sugita et al., 2003). For the items listed under geological environment conditions that may have an influences, the parameters shown in Table 2.1.2-5 need to be assigned as input data for the design (Kurihara et al., 2004).

Table 2.1.2-1 Relationship between overpack design and geological environmental condition（Revision of Kurihara et al., 2004）

design requirement	Influence of geological environmental conditions	Influence factor except geological environmental conditions	influence contents	Correspondence by design
containment of radionuclides in vitrified waste, corrosion resistance, pressure resistance	composition of groundwater, redox state of groundwater, temperature	Degradation of tunnel support material, chemical buffering property of buffer, heat production of vitrified waste	Corrosion behaviour, corrosion speed	･materials ･corrosion depth
	existence of a microbe, temperature	Proliferate and activity of microbe in buffer, heat production of vitrified waste	Corrosion behaviour, corrosion speed	･materials
	Groundwater hydrostatic pressure, ground pressure, rock mechanic property	Effective stress of buffer (creep behaviour of rock, corrosion expansion of overpack)	loading to overpack	･materials ･thickness for stress resistance
radiation shielding	Redox state of groundwater	Radiation from vitrified waste	Corrosion behaviour, corrosion speed	･thickness for radiation shielding

Table 2.1.2-2 Relationship between buffer design and geological environmental condition (Revision of Kurihara et al., 2004)

design requirement	influence of geological environmental conditions	Influence factor except geological environmental conditions	influence contents	Correspondence by design
low permeability	Composition of groundwater (ionic strength), hydraulic gradient, temperature	Heat production from vitrified waste	permeability	Material, specification
		Gas production from overpack corrosion	Radionuclide migration (influence by gas migration through buffer)	Evaluation as system
	Existence of fracture, groundwater flow velocity, temperature		Density of buffer（influence by extrusion of bentonite into the fracture）	
Colloid filtration, self-sealing ability	Composition of groundwater (ionic strength), temperature	Heat production from vitrified waste	Colloid filtration, swelling property	material, specification
	Existence of fracture, groundwater flow velocity, temperature		Density of buffer (influence by extrusion of bentonite into the fracture)	Evaluation as system
Mechanical stability properties	Composition of groundwater (ionic strength)	shape of waste package, weight	Mechanical support of the overpack	Material, specification
	Ground pressure, rock mechanic property	Evaluation of creep behaviour of rock, corrosion expansion of overpack and gas migration through buffer	Safety for shear destruction, mechanical support of overpack	Evaluation as system
Stress buffering properties	Ground pressure, rock mechanic property	Creep behaviour of rock, corrosion expansion of overpack	Loading to overpack	thickness
Thermal property	Temperature, thermal property of rock	Heat production from vitrified waste, overpack, tunnel support, buckfill	Degradation by heat production	Material, specification
Long-term behaviour	Composition of groundwater (pH etc.), temperature	Degradation of tunnel support material	Permeability, self-sealing property, etc.	Evaluation as system

Table 2.1.2-3 Relationship between geological environmental condition and design on backfill materials (Partial revision of the reference [Sugita et al., 2003])

design requirement	Influence of geological environmental conditions	Influence factor except geological environmental conditions	influence contents	Correspondence by design
Self-sealing ability for degradation of concrete support	Composition of groundwater (ionic strength) temperature	Degradation of tunnel support material	Self-sealing property	material, specification
Countermeasure against extrusion of backfill material into tunnels	Composition of groundwater (ionic strength) existence of fracture, groundwater flow velocity		Density (influence by extrusion of bentonite into the fracture)	material, specification
Low permeability at levels below an requirement in the tunne	Composition of groundwater (ionic strength), temperature	degradation of tunnel support material	Permeability	material, specification
Restraint of significant defomation against swelling pressure of buffer	Composition of groundwater (ionic strength, pH), ground pressure, groundwater hydrostatic pressure, rock mechanic property	Creep displacement quantity of rock, extrusion of buffer to the disposal tunnel	Stiffness	material, specification
No degradation of buffer		degradation of tunnel support material	Permeability	material, specification

Table 2.1.2-4 Relationship between geological environmental condition and design on waterproof plug (Partial revision of the reference [Sugita et al., 2003])

design requirement	Influence of geological environmental conditions	Influence factor except geological environmental conditions	influence contents	correspondence by design
Low permeability of plug itself	Composition of groundwater (ionic strength), temperature	Degradation of tunnel support material	permeability	material, specification
Low peameability or discontinuation of support and EDZ,	Composition of groundwater (ionic strength), rock strength, fracture frequency, EDZ		Hydraulic condition of plug and circumference	material, specification , slot

Table 2.1.2-5 The geological environmental data for design of engineered barrier system (Kurihara et al., 2004)

classification	input data of design
geochemistry	1)concentration of dissolved oxygen, Eh, pH 2) composition of groundwater • priority: $HCO_3^-/CO_3^{2-}/H_2CO_3$, SO_4^{2-}, HS^-/H_2S, Cl^-,CH_4, ionic strength (Na, Ca, K, Mg, anion) • others: P, NO_3^-, NH_3(aq), NH_4^+, B
mechanical	ground pressure, time dependent parameter, deformation coefficient, poison's ratio, unconfined compression strength, tensile strength
Hydraulic	Hydraulic gradient, groundwater hydrostatic pressure, groundwater velocity
Thermal	temperature, thermal conductivity, specific heat, geothermal gradient
other	microbe, conditions of fracture of the host rock

2.2 Applicability of design methods in the surface-based investigation phase of the Horonobe URL project

2.2.1 Geological environment conditions and disposal depth

Using information from the borehole investigations in the URL area and the proposed design of the underground facilities, the philosophy for determine the disposal depth and the way in which the design parameters were determined are described in this section. Investigations in Phase II and subsequent phases in the Horonobe URL project will be conducted in the URL facilities. This study will therefore address only a limited area around the shaft between boreholes HDB-3 and HDB-6 where the borehole investigations were conducted (Figure 1.1.1-2, several hundred square meters), in which URL will be constructed (Figure 2.2.1-1).

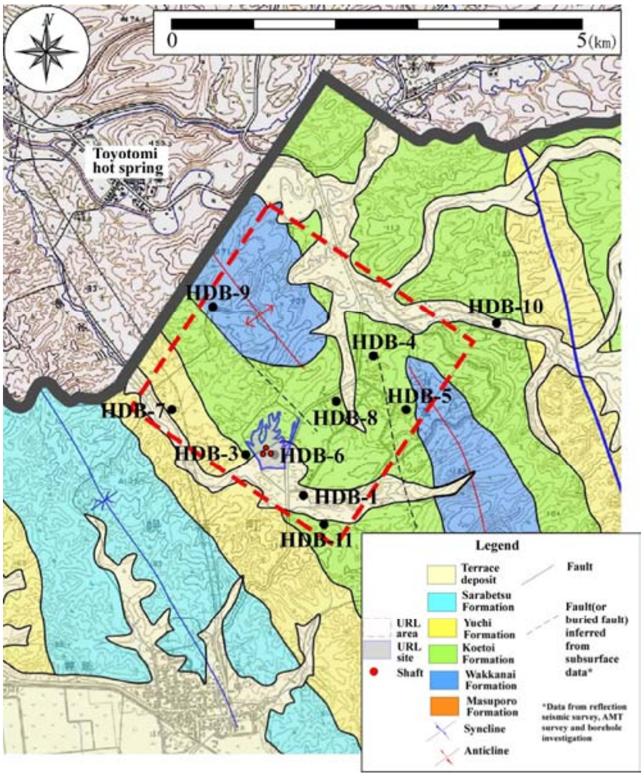


Figure 2.2.1-1 Horonobe URL construction area

(1) Determining the disposal depth

The H12 Report assumed a disposal depth of 1,000 m for hard rocks and 500 m for soft rocks based on four major perspectives: long-term safety, characteristics of the geological environment (e.g. geochemistry and flow characteristics of groundwater and mechanical properties of the rock mass), applicability of conventional construction and investigation technologies and the depth to be analyzed in the design (mechanical stability of underground excavations and thermal impact on the engineered barriers). The report concluded that the main factors determining disposal depth are the thermal impact on the engineered barriers for hard rock (relationship between the number of vitrified waste units per given area at the disposal depth and the temperature of the buffer) and the tunnel stability for soft rock (depth to be achieved using tunnel supports with reasonable thickness). It is stated in the Final Disposal Act that the disposal depth should be 300 m or more and, based on these considerations, the depth in this study was assumed to be between 300 and 500 m. The engineered barriers could be manufactured with given specifications and installed to achieve the required containment time regardless of the geological conditions if economic aspects are ignored. What is more conclusive for the decision on disposal depth is therefore whether any disturbances can be expected in the geological environment surrounding the engineered barriers. One such disturbance would be fracturing of the near-field rock. This would cause formation of mass transport pathways in the zone around the disposal tunnel or excavation disturbed zone (EDZ). Fractures could be propagated with time or develop anisotropic deformation, resulting in local changes in the buffer thickness. As a result, the mechanical stability of the near-field could be adversely affected (Figure 2.2.1-2). Such phenomena would have a large impact not only on the design of the engineered barriers, but also on the safety assessment of the repository and two requirements have therefore been set for the depth determination: mechanical stability of the disposal pits should be assured and a large homogeneous rock mass should be available in the vertical direction as an emplacement environment. The latter requirement was set because the horizontal heterogeneity in the geological properties of the Horonobe area is likely to be small on the scale of the underground facilities (only several hundred square meters). Also, mechanical stability can be assured for horizontal disposal tunnels. Although the stability depends on the balance between the mechanical properties and initial stress state of the rock, it would be possible to choose a direction in which the extent of EDZ is at a minimum if the direction of the initial stress is known. Therefore, a depth that meets the requirement described above was determined based on the vertical distribution of the rock properties. Figure 2.2.1-3 shows rock mass classifications at the URL site, based on the evaluation made in the design of the Horonobe underground research facility. Table 2.2.1-1 shows the deformation and strength for each classification. As seen from these data, rock at a depth of about 450 m, or more precisely the rock classified as CM-H over a section of 40 m between depths of 434 m and 474 m, has proven to have sufficient thickness and properties to satisfy the mechanical stability requirements of the disposal pits (see section 2.2.3 for details). Further studies were therefore conducted on the geology at a target depth of 450 m. The methods for determining the values shown in Table 2.2.1-1 are described in section 2.2.1 (2).

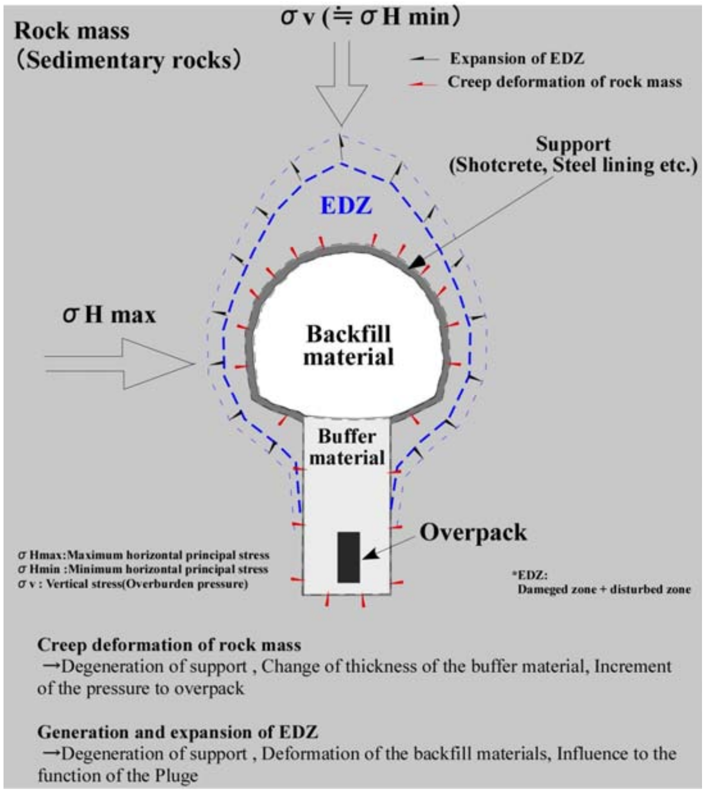


Figure 2.2.1-2 An influence factor for a design of a disposal system

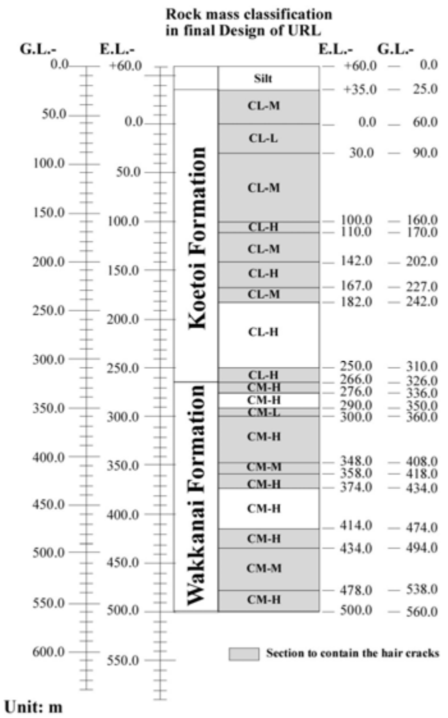


Figure 2.2.1-3 Rock mass classification applied to a design of Horonobe URL

Table 2.2.1-1 Deformability and strength of rock mass classification

Rock mass classification	Deformability	Strength		
	E [MPa]	C [MPa]	ϕ [°]	σc [MPa]
CM-H	2,500	5.2	25.0	16.3
CM-H(Hr)	2,000	1.6	25.0	5.0
CM-M	1,500	3.1	25.0	9.7
CM-M(Hr)	1,350	1.9	25.0	6.0
CM-L	500	1.0	25.0	3.1
CM-L(Hr)	500	1.0	25.0	3.1
CL-H	1,300	2.2	15.0	5.7
CL-H(Hr)	1,040	1.5	15.0	3.9
CL-M	500	0.8	15.0	2.1
CL-M(Hr)	450	0.6	15.0	1.6
CL-L	300	0.5	15.0	1.3
CL-L(Hr)	300	0.5	15.0	1.3

*(Hr):This means that hair crack affect the mechanical properties

(2) Characterizing the geological environment at the depth of interest

The report on the surface-based investigations at the Horonobe URL site (see separate volume, on geoscientific research, sections 4.3.3, 4.4.4, 4.5.3. and 4.6.3) overviews the geological environment on the west side of the Omagari Fault as follows.

- The formations up to approximately 700 m depth can be roughly divided vertically into three different zones in terms of geology and properties. The changes between zones are continuous (see Figure 2.2.1-4 in the geoscientific research report).

- There are two types of fractures in the rock, vertical or horizontal to the bedding plane, which will have a certain influence on rock permeability but not much on mechanical stability. The permeability of siliceous mudstones tends to decrease with increasing depth.

- No confined groundwater is observed and in general, hydrostatic pressure only varies with depth. Regarding the initial stress, most of the vertical maximum principal stresses are in an E-W direction, and the maximum ratio between maximum and minimum stresses in the horizontal plane is around 1.5.

- The concentration of dissolved components in the groundwater tends to increase with increasing the depth.

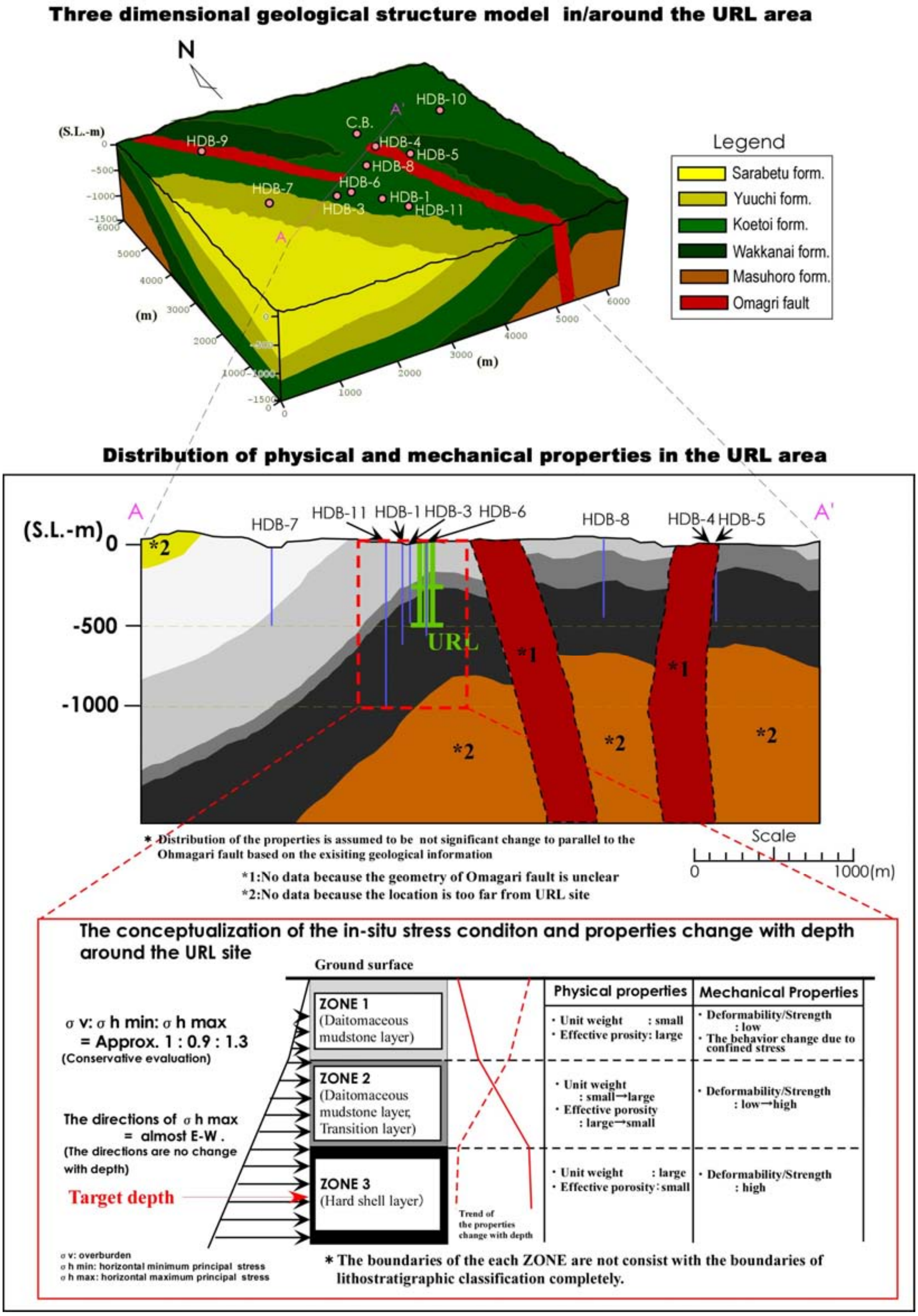


Figure 2.2.1-4 Conceptual model on rock mechanics

Input data for the design of the engineered barriers shown in Table 2.1.2-5 were determined based on borehole investigations and laboratory tests using samples from boreholes. The parameter values at the depth of interest (around 450 m) were calculated considering property correlations between each zone. For consistency reasons, the values for mechanical properties were taken from those defined in the rock classifications for the Horonobe URL design.

Three zones were defined - Zone 1, Zone 2 and Zone 3 - on the west side of the Omagari Fault according to the geological boundary between the Koetoi and Wakkanai Formations and based on the conceptual mechanical model. Zone 1 consists of diatomaceous mudstones (Koetoi Formation), Zone 2 is the transition zone between the Koetoi Formation and Wakkanai Formations, each extending about 50 m upwards and downwards from the boundary (total thickness of around 100 m) and Zone 3 consists of siliceous mudstones (Wakkanai Formation). Since the Horonobe URL design assumed that the boundary between the Koetoi Formation and the Wakkanai Formation was located at a depth of around 325 m, the lower limit of Zone 1 (boundary between Zone 1 and Zone 2) at this location would be at a depth of approximately 275 m and the lower limit of Zone 2 (boundary between Zone 2 and Zone 3) at a depth of approximately 375 m. The depth of each boundary was determined for each borehole using the results of geophysical logging and the depth profile of the properties within each zone was then evaluated (see sections 4.6.3 and 4.10.4 of the separate volume on geoscientific research). The results basically showed a linear distribution for each property with some variability, except for the permeability and chemistry of the groundwater. Therefore, linear interpolation formulae were formulated for the distribution patterns within individual zones to determine the property values at the depth of interest (Matsui et al., 2005a).

An example of determining the geological environment conditions at a depth of 450 m (Horonobe 450 m level) is shown below (Matsui et al., 2005a). There are certain limitations on estimating the range of geological conditions deep underground in detail because the estimation is based only on data obtained from a limited number of borehole investigations during the surface-based investigation stage. Therefore, information on the geological environment required for designing the engineered barriers should be improved using knowledge obtained during construction and investigations during the construction/in-situ test phase. With such information, the values to be used for analysis should be revised in phases.

In the following subsections, the principles for determining properties at the depth of interest and the results for physical and mechanical properties, initial stress, thermal properties, groundwater hydrochemistry and hydraulic properties are presented. The relationship between the design items and the properties used for their analysis is shown in Table 2.2.1-2.

Table 2.2.1-2 Relations with design input data and design item

Design input data	design item
Physical property (density, effective porosity))	・Estimation of disposal tunnel spacing and disposal pitch from the viewpoint of heat influence ・Evaluation of gas migration behaviour
Mecanical property (modulus of elasticity coefficient, unconfined compressive strength, poison's ratio, cohesion, angl of internal friction, etc.,)	・Design of the disposal tunnel and disposal pit ・Estimation of disposal tunnel spacing and disposal pitch from the viewpoint of mechanical influence ・Evaluation of long-term mechanical stability of the host rock
Initial stress	・Design of the disposal tunnel and disposal pit ・Estimation of disposal tunnel spacing and disposal pitch from the viewpoint of mechanical influence ・Evaluation of long-term mechanical stability of the host rock
Thermal property (geothermal gradient, thermal conductivity, specific heat, etc.)	・Estimation of disposal tunnel spacing and disposal pitch from the viewpoint of heat influence ・Evaluation of gas migration behaviour
Geochemistry (pH, Eh, ionic strength, etc.)	・Design of the buffer ・Design of the overpack ・Evaluation of long-term mechanical stability of the buffer ・Evaluation of extrusion and erosion behaviour of buffer ・Design of the backfill ・Design of the plug
Hydraulic property	・Design of the buffer

1) Determination of physical properties (density, effective porosity)

The depth profile for density is shown in Figure 2.2.1-5 and that for effective porosity in Figure 2.2.1-6. Correlations shown in the figures were formulated based on the data obtained from the borehole investigations in Horonobe. The property values at the Horonobe 450 m level used for the analyses were determined as 1.89 Mg m^{-3} for wet density and 39.7% for effective porosity. The depth from the upper boundary of Zone 3 to the Horonobe 450 m level is 75 m. Although it is not a parameter required for the design, the effective porosity was determined as a reference, because it is related to the mechanical and thermal properties.

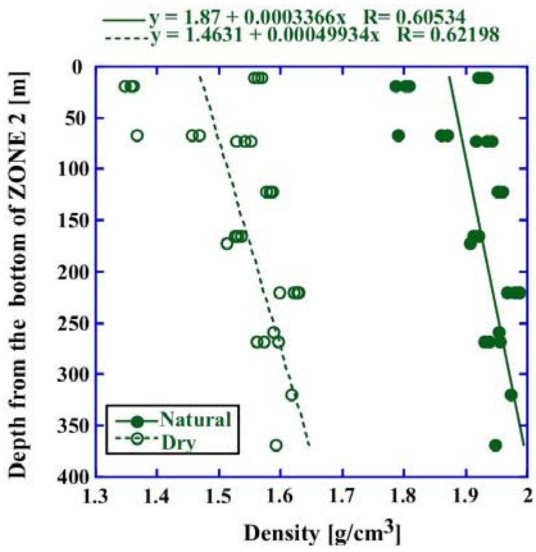


Figure 2.2.1-5 Distribution of the density of the rock

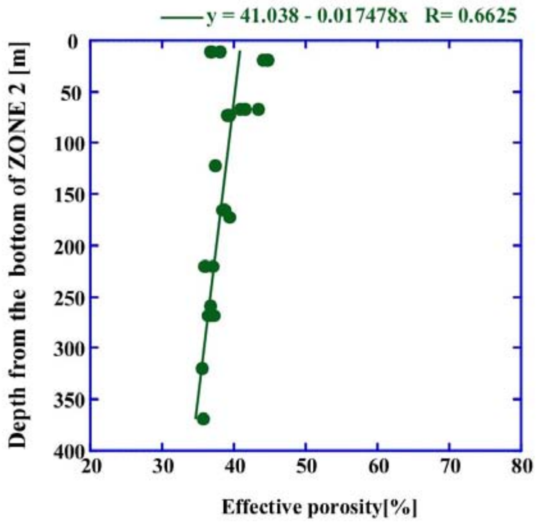


Figure 2.2.1-6 Distribution of effective porosity of the rock

2) Determination of mechanical properties

An example of setting mechanical properties for the Horonobe URL design is presented below for rock strength (cohesion C, internal friction angle φ), elastic coefficient E, Poisson's ratio υ, tensile strength T and initial stress. The values were determined according to the procedure shown in Figure 2.2.1-7 (see section 5.2.4 in the volume on geoscientific research). The data obtained from investigations in boreholes HDB-3 (Yamamoto et al., 2003) and HDB-6 (Yamamoto et al., 2004a), which were the nearest boreholes to the proposed Horonobe URL, were used. It should be noted that anisotropic deformation and strength due to the effects of sedimentation are often observed in sedimentary rocks. For the diatomaceous mudstones and siliceous mudstones at Horonobe, laboratory tests showed minimum values for the mechanical properties in the vertical direction and the test results for the vertical direction were therefore used as the mechanical properties (Matsui et al., 2005a).

① Rock mass classification

- Categorize the boring cores based on the fracture frequency and its hardness in HDB-3 and HDN-6
- Inspect the hair crack in every 10m length in HDB-3 and HDB-6 (see Fig. 2.2.1-9)

② Model construction of host rock

- Construct the columnar section on rock mass classification at URL position (Fig 2.2.1-3)

③ Mechanical parameter

Figure 2.2.1-7 Outline of mechanical parameter setting for the Horonobe URL design

In the method shown in Figure 2.2.1-7, a model was constructed using data from boreholes HDB-3 and HDB-6, and the geological properties in the shaft located between these boreholes (Figure 2.2.1-1) estimated based on the model. In the case of the disposal project, a similar method will be used to estimate the geological conditions deep underground where the disposal tunnels would be constructed. For the detailed design of the access tunnels, however, the area of interest will be larger than the URL. This would make it necessary to conduct borehole investigations in the vicinity of the planned access tunnels to provide sufficient data ensure that the uncertainties and risks associated with the construction of the first access tunnel, for which available information is very limited, could be minimized.

In the following paragraphs, the step shown as (d) in Figure 2.2.1-7, determination of different mechanical properties of the rock, is shown in more detail. With regard to Poisson's ratio ν, the static Poisson's ratio was calculated from the result of unconfined compression tests and the average values for each zone were determined as the Poisson's ratio. Unconfined tensile strength was determined as being 1/10 of the converted unconfined compressive strength σ_c that was calculated using the equation (2.2.1-1) for the Mohr-Coulomb failure criterion with the cohesion C and internal friction angle φ shown in Table 2.2.1-1 (see section 4.6.3 in the volume on geoscientific research).

$$\sigma_c = \frac{2 \cdot C \cdot \cos\phi}{1 - \sin\phi} \qquad (2.2.1\text{-}1)$$

The methods for determining cohesion C, internal friction angle φ and elastic coefficient E are shown in Figure 2.2.1-10. The values for each rock classification determined using these methods are shown in Table 2.2.1-1. Table 2.2.1-3 shows the design-based properties at the Horonobe 450 m level determined using the method described above. The residual strength was determined in the same way as for the peak strength, i.e. based on the results of triaxial compression tests (CD). With regard to time-dependent parameters, constants n and m in the compliance variable constitutive equation were determined for the Wakkanai Formation as part of the research on long-term mechanical deformation of the rocks (Okubo, 2004, 2005).

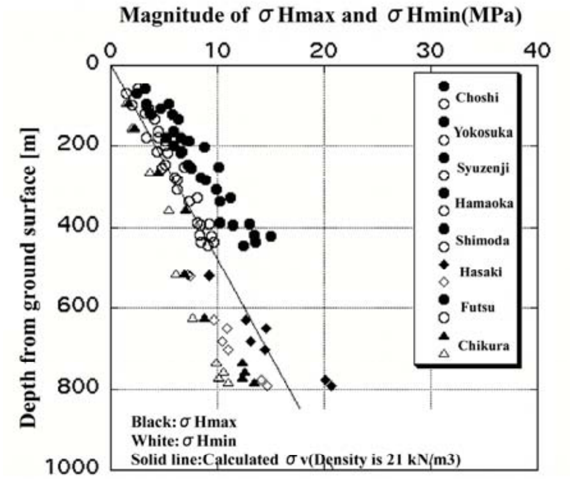
Table 2.2.1-3 Dataset of mechanical properties for design at 450m depth of Horonobe

Target depth [m]	Rock mass classification	Modulus of elasticity E[MPa]	unconfined compressive strength σ_c [MPa]	poison's ratio v[-]	Peak strength		Residual strength		tensile strength [MPa]
					cohesion c [MPa]	internal friction angle φ[°]	cohesion c[MPa]	internal friction angle φ[°]	
450	CM-H	2,500	16.3	0.186	5.2	25.0	1.27	29.8	1.63

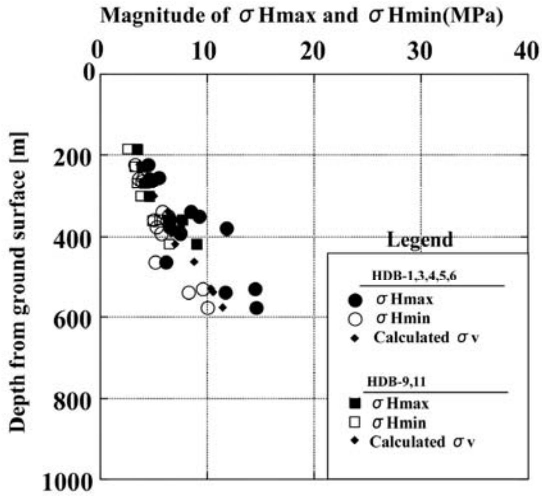
For the initial stress, the average of the values measured in boreholes HDB-3 and HDB-6 using the hydro-fracturing method was used as shown in Table 2.2.1-4. The overburden pressure calculated from the unit weight was used as the vertical stress S_v because it is almost the same as the measured initial stress. The depth gradient of the principal stress shown in Table 2.2.1-4 is consistent with the trend observed from the results of hydro-fracturing measurements conducted in other areas of Japan (Figure 2.2.1-8). The maximum horizontal principal stresses (S_{max}) were generally in the E-W direction. Considering the overburden pressure as vertical earth load S_v, it can be assumed that the initial stress ratio will remain almost constant regardless of depth (Matsui et al., 2005b).

Table 2.2.1-4 Measurement results and estimation value of initial stress

	Depth [m]	S_{max} / S_v	S_{min}/ S_v		Depth [m]	S_{max} / S_v	S_{min}/ S_v
HDB-6	223.0	1.21	0.91	HDB-6	529.5	1.59	1.08
	256.0	1.10	0.87		539.5	1.01	0.76
	338.1	1.56	1.06		576.0	1.47	1.00
	351.0	1.71	1.13	HDB-3	263.5	1.02	0.80
	416.0	0.85	0.72		392.5	1.20	0.90
Estimation value	S_v:S_{max}:S_{min}＝1:1.3:0.9, horizontal stress ratio:1.4, S_v＝overburden pressure, S_v(450)=7.21MPa						



Change of the magnitudes of σHmax and σHmin with depth measured at eight different locations in Japan
(Density of rock is 20 to 22 kN/m3)



Change of the magnitudes of σHmax and σHmin with depth measured in HDB-1 ～HDB-11 in study area

Figure 2.2.1-8 Comparison of the stress measurement results by hydraulic fracturing method using rock sample in Japan
and stress measurement results in the deep borehole (Niunoya et,al., 2006)

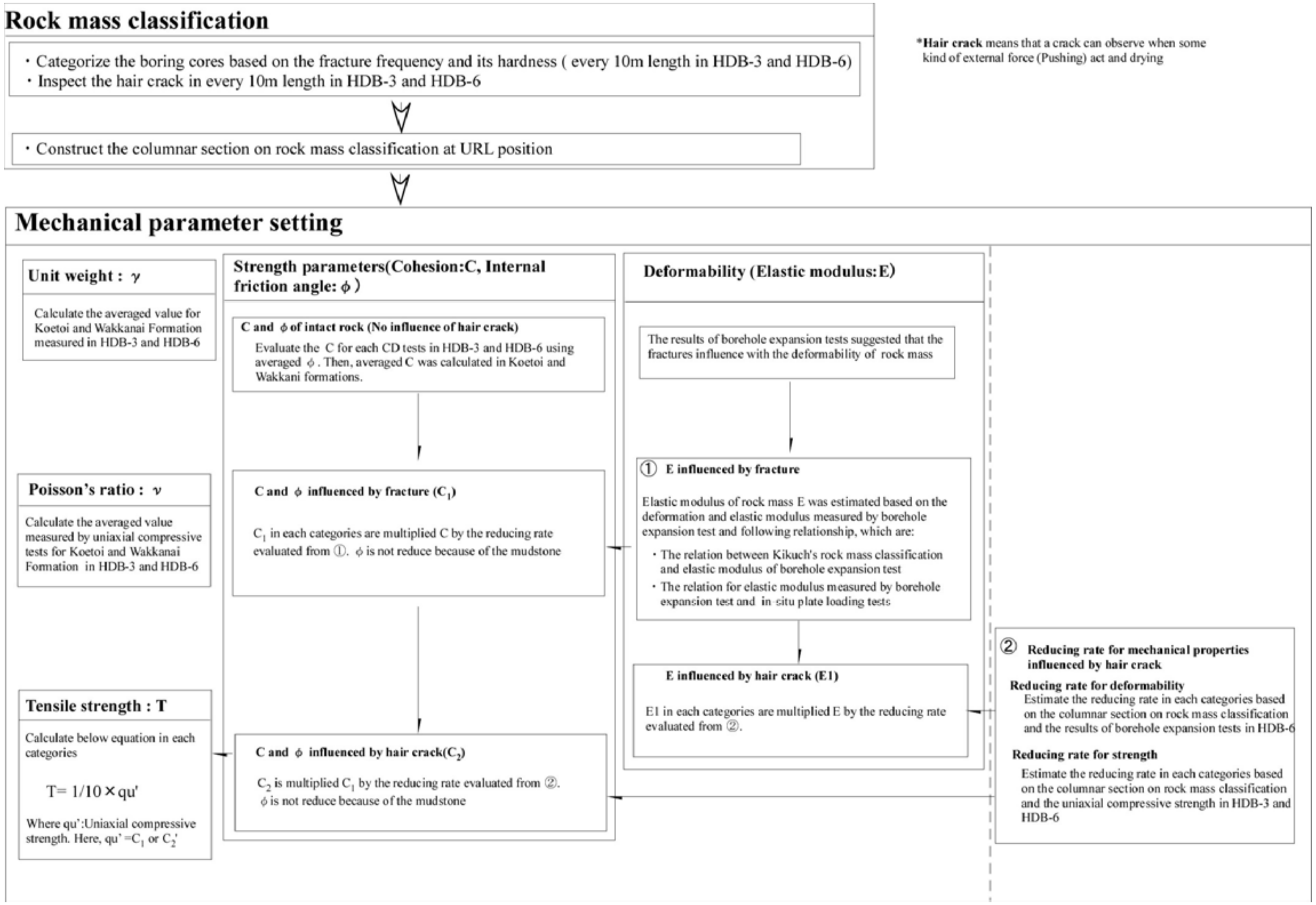


Figure 2.2.1-9 Influence of fracture and hair crack to the mechanical properties of the host rock

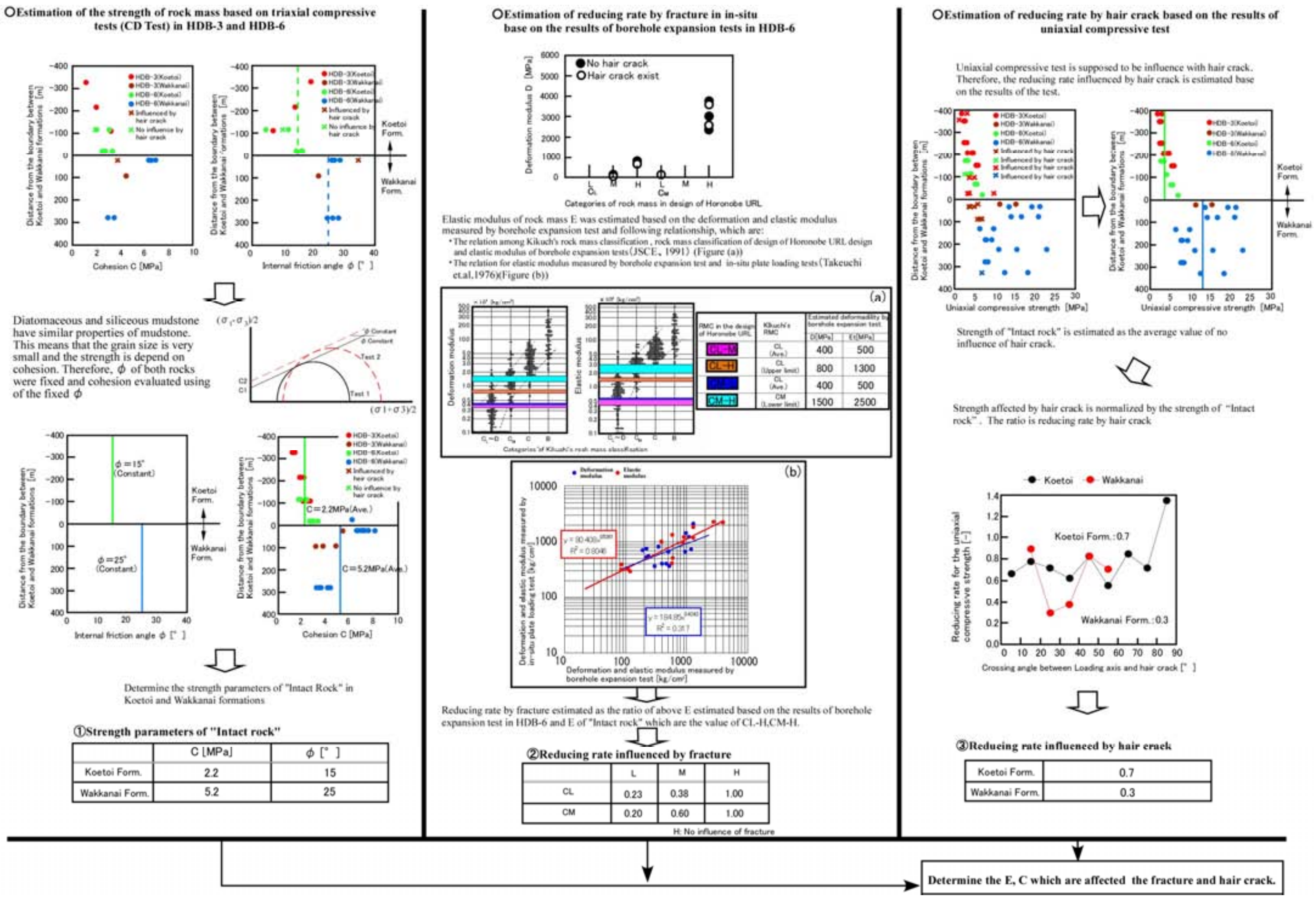


Figure 2.2.1-10 Determination procedures of parameters of the rock such as cohesion, internal friction angle and modulus of elasticity

3) Determination of thermal properties

The data on the geothermal gradient were obtained from thermal logging and the long-term monitoring in boreholes HDB-1 (Yamamoto et al., 2002), HDB-3 and HDB-6 (Matsui et al., 2005a). The thermal logging was, however, conducted after drilling had reached a certain depth and directly after drilling and the measurement results do not necessarily reflect the exact temperatures of the rock and should be handled as relative values. Compared to this, the data obtained using the long-term monitoring equipment (MP system) are more reliable. Figure 2.2.1-11 shows the data obtained using the MP system, where y is the temperature (degrees C) and x is the depth (m) in the equation representing the correlation in the figure. The temperature-depth correlation is expressed here using a first-order linear approximation, assuming that groundwater flow would be slow at the URL site and there should be no heat sources which would have an influence on the geothermal gradient. Although a difference can be seen between the data from boreholes HDB-1 and HDB-3 in the figure, the data from HDB-3 were used because the temperature of shallow well water measured near the surface remained constant between 7°C and 9°C throughout the year (Matsui et al., 2005a). Due to the limited data available, however, it would be necessary to validate these hypotheses using data obtained from further monitoring.

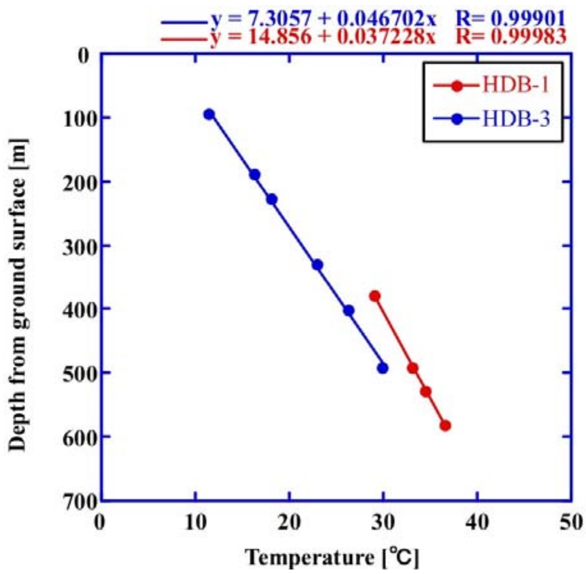


Figure 2.2.1-11 Measurement results of geothermal gradient by MP system

Thermal conductivity, specific heat and linear expansion coefficient were determined separately for Zone 1 and for Zones 2 and 3 because of the different lithofacies. They were classified as those at temperatures lower and higher than around 60°C because previous studies indicate that the thermal properties change at around 50 to 60°C (Yamamoto et al., 2004b). The data were grouped according to rock conditions into those under saturated, natural or dry conditions, as well as according to depth, and correlations were derived from these data (Matsui et al., 2005a). For the saturated conditions with temperatures lower than 60°C (Figure 2.2.1-12 and 2.2.1-13), the correlation at the temperature nearest to that of the rock at the depth of interest (28.3°C) was selected based on the temperature obtained from the geothermal gradient as mentioned above. The thermal conductivity value was determined with the correlation at 25°C, the specific heat at 30°C and the linear expansion coefficient at 40°C (Matsui et al., 2005a). It should be noted that the thermal conductivity may sometimes have a negative value for Zone 1. One possible reason is the evaporation of the water absorbed in minerals, but this is not yet clear. If thermal loading can be expected in this zone, more detailed studies would be required.

The values determined for the thermal properties are summarized in Table 2.2.1-5.

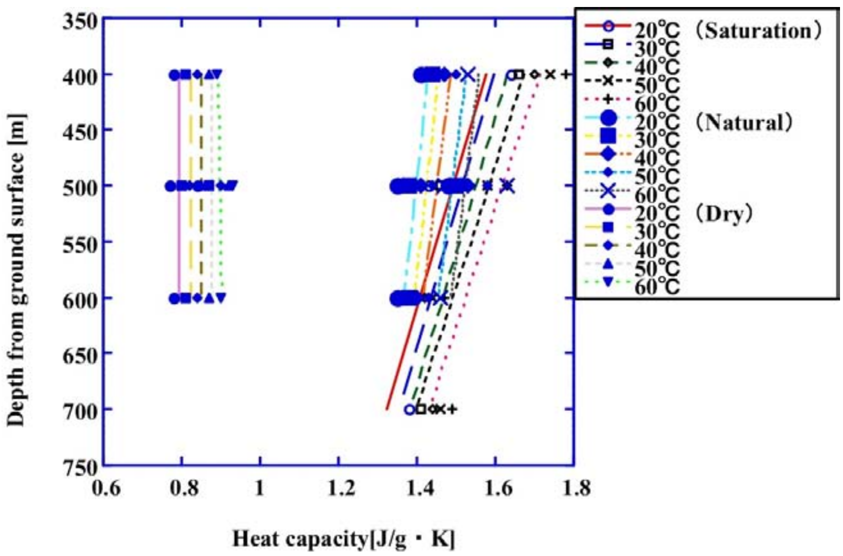


Figure 2.2.1-12 Distribution of specific heat on Zone 2 and 3 (60 ºC below)

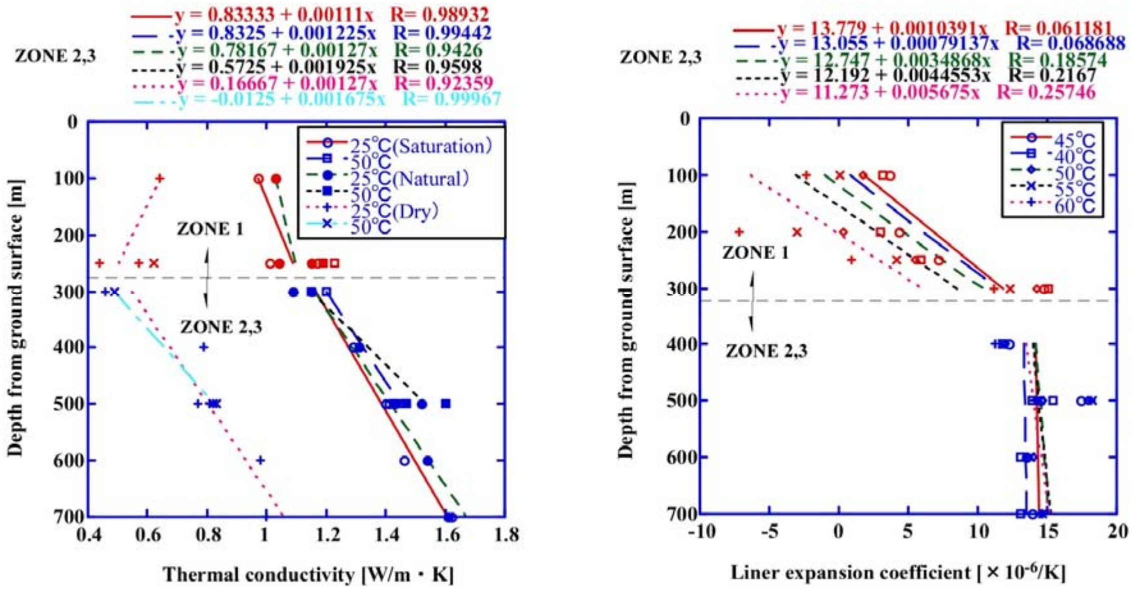


Figure 2.2.1-13 Distribution of thermal conductivity and coefficient of linear thermal expansion on the Zone 2 and 3 (below 60 ºC)

Table 2.2.1-5 Estimation of thermal properties at target depth (450m)

geothermal gradient	・Geothermal gradient:4.7°C/100m ・geothermal at 450m depth:28.3°C
thermal conductivity	1.33 W m^{-1} K^{-1} (calculation by correlative expression at 25°C)
specific heat	1.56 kJ kg^{-1} K^{-1} (calculation by correlative expression at 30°C)
Liner expansion coefficient	13.41×10^{-6} K^{-1} (calculation by correlative expression at 40°C)

4) Determination of groundwater chemistry (see section 4.4 in the separate volume on geoscientific research)

The groundwater chemistry to be used as input for the design of the engineered barriers for the URL site at Horonobe was determined based on analysis data for the groundwater sampled from borehole HDB-6 (approximately 600 m depth), which is located nearest to the proposed URL site. The groundwaters analyzed to date can be classified into two types: so-called formation water that is pumped up from underground formations and core water that is squeezed from cores. The samples from borehole HDB-6 included two formation waters and nine core waters obtained from different depths. The core waters were obtained using an unconfined compression type squeezing apparatus that applies pressures up to 70 MPa to squeeze out water. The properties of the core water may differ from those of the actual groundwater (formation water) due to contact with air, mineral dissolution and/or precipitation in the core and other factors occurring during the squeezing process (e.g. Pearson et al., 2003). The data should be therefore treated with caution. A comprehensive evaluation was made of the data on the HDB-6 groundwater from different aspects including knowledge of the depth dependence of groundwater chemistry, the results of a thermodynamic analysis of groundwater chemistry and information on the minerals found in the formations in the Horonobe area. The groundwater chemistry estimated in this way for a depth of 450 m was used as the design basis for the Horonobe 450 m level as shown in Table 3.3.3-1 (Matsui et al., 2005a). The estimation was made for 25°C using the geochemical analysis codes PHREEQC (Parkhurst, 1995) and the thermodynamic database (Yui et al., 1999). It should be noted that the determination was based only on the current geological environment and chemical composition of the groundwater. An evaluation taking the potential long-term evolution would be a future task.

5) Determination of hydraulic properties (see section 4.3 in the separate volume on geoscientific research)

Hydraulic gradients for each element obtained from the groundwater flow analysis carried out by JNC generally show values smaller than 0.02 in the zone including the 450 m level (around E.L. -500 m) (Kurikami et al., 2005). This analysis also indicates that, the greater the depth, the more the flow in a south to southwest direction prevails and the average of the hydraulic gradients at 400 m intervals with boreholes HDB-3 and HDB-6 as a center near the URL site in the NE-SW direction shows that the average hydraulic gradients for the zone with a width of 100 m including the depth of interest (around G.L. -450 m) are as small as between 0.007 and 0.01. The depth profile of the hydraulic gradients at the center of the URL site (center of three shafts) (Figure 2.2.1-14) shows values around 0.01 in the zone near E.L. -500 m, which is deeper than 450 m. Therefore, the hydraulic gradient was determined as being 0.01 in this study. For the hydraulic conductivity, the groundwater flow analysis uses an equation in which the characteristics of the Wakkanai Formation, i.e. depth dependence of the hydraulic conductivity and anisotropic permeability caused by the geometric characteristics of fractures, are taken into account and which therefore should best explain the distribution of hydraulic conductivities in the Wakkanai Formation. However, the rocks at a depth of 450 m (see section 3.2) are known to show almost no influence of small fractures and the results of in-situ tests also showed that the rock permeability in this zone showed no dependence on depth. The permeability decreases with increasing depth, but is independent of fracture frequency (Kurikami et al., 2005). Assuming that the properties are not anisotropic depending on the geometric characteristics of fractures, another equation (2.2.1-2) was therefore used in this study, which can represent the permeability of the whole Wakkanai Formation, including the depth dependence (Kurikami et al., 2005):

$$\log(k) = -0.0105z-3.9118 \tag{2.2.1-2}$$

where z is the depth from ground surface. The hydraulic conductivity k is 2×10^{-9} m s^{-1} for the depth of interest (around G.L.-450 m).

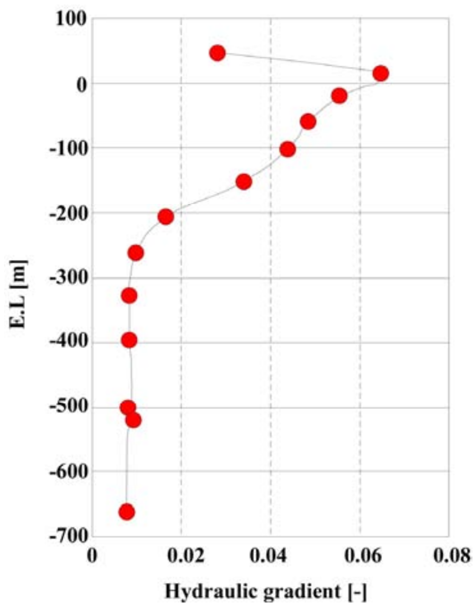


Figure 2.2.1-14 Distribution of hydraulic gradient at center of Horonobe URL facility

2.2.2 Influence of the geological environmental condition on the Horonobe URL project

Points that need to be taken into account when applying the design methods are listed in Table 2.2.2-1, taking the features of the geological environment at Horonobe as an example (Matsui et al., 2005a). The most important features of the geological environment in Horonobe are that (a) it consists of soft sedimentary rock*, (b) saline groundwater occurs and (c) it bears methane gas. These features were the basis for the studies made on the tunnel stability (design of support), disposal tunnel spacing and waste package emplacement pitch, from a mechanical and thermal viewpoint, and for the case study design of the engineered barriers and the backfill taking the

* There is no strict definition of soft rock. An assessment report (Japan Society of Civil Engineers, 1996) states that mudstones or sandy mudstones are defined as soft rock if they have an unconfined compressive strength between 10 and 100 or lower than 200 kgf/cm^2. Accordingly, the term soft rock here means rock with a unconfined compressive strength of 20 MPa or lower.

influence of the saline groundwater into account. The results of the studies are presented in the following sections.

Table 2.2.2-1 Consideration point of applicability of design procedures on the geological environmental conditions of Horonobe

item	characteristic of geological environment of Horonobe	Consideration point of applicability of design procedure
geological structure	・Target depth : 450m（434～474m） ・distance of target host rock - vertical distance : 40m - horizontal distance: estimation of continuously distance along geological structure about URL facility scale ・There is hostrock changing transitionally from siliceous mudstone/diatomaceous mudstone to diatomaceous mudstone ・consideration of existence of dislocation（not confirm it directly）	
Rock mechanics (strength and stress)	・Rock mass classification：CM-H ・direction of major horizontal principal stress：east-west direction ・anisotropically stress condition (vertical direction : 1.0, minimum horizontal principal stress : 0.9, maximum horizontal principal stress : 1.3） ・continuously deformation of host rock under the constant stress condition and hydro-mechanical couple behaviour ・anisotropy of strength property, deformation property and physical property	・Necessity or not of the construction of support into the disposal pit ・Evaluation of mechanical stability of tunnel ⇒consideration of support material（applicability of low alkaline concrete support） ・Estimation of disposal tunnel spacing and disposal pitch from the view point of mechanical influence ・Seismic stability assessment ・Evaluation of long-term mechanical stability of the host rock(reflection of design of the buffer and overpack) ・Evaluation of hydro-mechanical couple behavior（confirmation of designed support and rock mechanics stability） ・set of physical parameter consider anisotropic
Thermal property	・geothermal gradient：4.7 °C/100m ・temperature at target depth (450m)：about 28 °C ・thermal property of the host rock(thermal conductivity : 1.33 W m^{-1} K^{-1}, specific heat : 1.56 kJ kg^{-1} K^{-1}) ・anisotropic	・Estimation of disposal tunnel spacing and disposal pitch from the view point of heat influence（confirmation of specification of engineered barrier）
Hydraulic property	・setting of hydraulic permeability : 2×10^{-9} m s^{-1}・hydraulic gradient : 0.01 ・distribution of hydrostatic pressure ・The direction of groundwater flow estimated south or southwestern flow. ・A big confined aquifer zone is not confirmed	・design of the buffer
Geochemistry	・concentration of chloride ion is 0.1 ~ 0.4M ・pH : approximately 6～7 ・Total carbon is less than 0.1M ・The concentration of carbonate is less than 0.1M	・The design that considered influence to performance of the buffer, backfill and plug ・Influence to corrosion resistance of overpack materials
other	・Existence of methane	・Evaluation of gas ejection volume (enlarged prevention of an accident, secure of ventilation. secure of a evacuation route

2.2.3 Applicability of the design method for the underground facilities

Of the steps shown in Figure 2.1.1-2, evaluation of the mechanical stability of underground tunnels and design of the disposal tunnel spacing and waste package emplacement pitch are related to the design of the underground facilities. For these items, a design method based on the surface-based investigations at Horonobe and the case study design using design-base data on the geological properties at the Horonobe 450 m level shown in Tables 2.2.1-3 to 2.2.1-5 are described below (Matsui et al., 2005a).

(1) Design methods based on the surface-based investigations

Of the design methods applied in the Horonobe URL (Matsui et al., 2005a), those related to the underground facilities were selected and compared with those described in the H12 Report, as an example of design using actual geological data. In the course of this study, important points and issues to be addressed in the future were identified for the design of underground facilities in sedimentary rock.

1) Procedures

The evaluation of the mechanical stability of tunnels and the tunnel support design were conducted in three steps in the H12 Report as follows (JNC, 1999).

Step 1: Define the specifications of the support for each tunnel based on theoretical analyses.

Step 2: Validate the support specifications defined by theoretical analyses by numerical analysis, modeling the access tunnel as a single tunnel and the other tunnels as closely connected tunnels, and defining the disposal tunnel spacing and waste package emplacement pitch.

Step 3: Check the seismic resistance of the tunnels with the support specifications defined in Steps 1 and 2 by numerical analysis to define the extent of reinforcement required for the intersections of the tunnels.

The evaluation of tunnel stability in step 1 of the design of the Horonobe URL was conducted by drawing up reference patterns for the support incorporating rock bolts based on the available information about rock properties using an empirical approach, taking the following issues into consideration.

- The URL site has been selected and information on its mechanical characteristics (rock properties and initial stress) is available.

- The design is not for generic evaluations such as those made in the H12 Report that aim to demonstrate the feasibility of construction.

- Construction of underground facilities generally uses rock bolts as one of the key support elements, but there is as yet no standard modeling approach.

In Steps 2 and 3, the tentative pattern prepared in Step 1 will be verified by numerical analyses (two-dimensional elastic-perfectly plastic analysis) because very few underground facilities have been constructed at depths as great as 500 m from the ground surface, which would rule out the empirical method alone for assuring tunnel stability. The integrity of the tunnel intersections was evaluated after a three-dimensional elastic analysis had been made for tunnel geometries and layout in which three-dimensional stress was re-allocated onto existing supporting elements.

2) Indices for tunnel stability

Table 2.2.3-1 shows the tunnel stability indices used for the Horonobe URL, together with the differences from those of the H12 Report. The reasons for these differences are described below.

Table2.2.3-1 Comparison of estimation indicator of cavity stability

Indicator	H12 Report	Horonobe URL
(a) Support stress intensity	Within admissible stress	Within admissible stress
(b) Stress condition	Being area that domain that local safety factor is less than 1.5 can be improved by countermeasure construction	Being area that plastic field can be improved by countermeasure construction（lock bolt）
(c) Deformation（normal strain and maximum shear strain of rock）	Being area that domain that exceeds equation (2.2.3-1) in figure 2.2.3-1 (median value) can be improved by countermeasure construction	Indicator for informatization construction

The indices shown in Table 2.2.3-1 are (a) stress intensity of the support, (b) local safety margin of the rock and (c) normal strain and maximum shear strain of the rock (Figure 2.2.3-1). Thanks to progress in analysis technologies, complex and detailed design for the support is straightforward nowadays. Usually, displacement measured during construction is used to re-evaluate the tunnel stability and the result is then reflected accordingly in the revision of the design-base specifications. In this sense, index (c) is becoming more important in so-called intelligent construction. Index (b) is used only as qualitative criterion to decide which construction measures should be taken in which zone. The final decision on tunnel stability is made using index (a). Tunnel stability before installing the support (e.g. risk of rock fall) is very important for the safety of construction work. Following the Horonobe URL design, in which the above points were taken into account, support for a single tunnel and a single disposal pit in the repository will be designed based on two criteria: (a) checking the support integrity using the allowable stress intensity design method and (b) the allowable limit for the width of the plastic zone calculated by elasto-plastic analysis (extent of the plastic zone in the rock, which can be properly supported by conventional rock bolting).

Design and construction on such a large scale with 50 tunnels in a single panel, a feature specific to geological disposal, have not been seen in existing underground facilities and the validity of the design has not been demonstrated. Therefore, a certain safety margin should be included in the evaluation and the design. The indices shown in Table 2.2.3-2 were used in this study. However, it will be necessary to include criteria related to rock creep behavior, the influence of hydrostatic pressure deep underground and structural fracture mode, and to constantly verify the design method and indices.

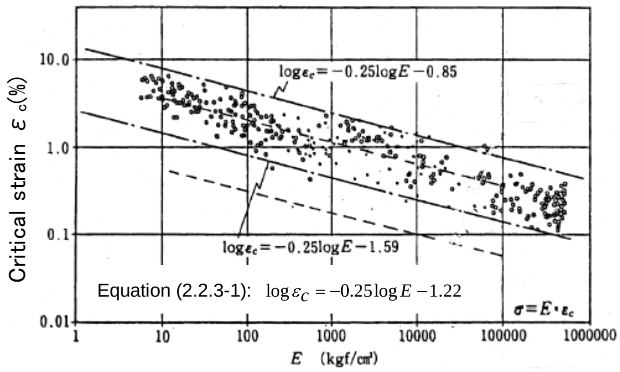


Figure2.2.3-1 Relationship between critical strain and modulus of elasticity (Sakurai and Adachi,1988)

Table2.2.3-2 Estimation indicator of cavity stability for tunnel spacing and disposal pit pitch

1	For pillars, domain more than local safety factor 1.5 should be secured enough.
2	Tunnel spacing should be secured more than 2 times of excavation width at least.
3	Support stress intensity should be secured safe degree of indispensability to design strength of support materials.
4	Maximum shear strain of area around tunnel should not change greatly by making tunnel spacing small.

3) Design considering geological environment conditions

As described in section 2.2.1 (2), the rocks at the Horonobe URL site have the mechanical characteristics of sedimentary rocks classified as soft to medium-hard rocks in an engineering sense, with anisotropic stress conditions, porous and anisotropic petrographic properties. Table 2.2.3-3 shows the additional issues to be addressed in designing a repository in soft sedimentary rocks, as identified in the course of the Horonobe URL design.

Table2.2.3-3 Items that need newly examination
in an facility design of sedimentary rock

1	Studies about stability when stress state of rock is anisotropic
2	Studies about long-term deformation behavior of rock for construction and operation period
3	Studies about water-stress coupled mode just after excavating that should be considered when it is porous rock
4	Studies about setting method of property for analysis that considered anisotropy of rock

Matsui et al. (2005a) carried out case studies on items 1 to 3 shown in Table 2.2.3-3 for the Horonobe URL, and confirmed that support design based on an elastic-perfectly plastic analysis and its construction plan are unlikely to cause problems. Item 4 could be dealt with by making a conservative design if the minimum values and their directions are known, even for the case with anisotropic properties. The design in this study also took into consideration anisotropy for the mechanical properties for which data obtained from the borehole investigations indicated anisotropy. Three-dimensional analyses simulating excavation processes should be conducted for the disposal tunnel and disposal pit because complex anisotropic stress conditions are expected. The results from these analyses will be verified using data obtained during the construction and operation phases of the URL project.

A new concept for design and construction was applied in the Horonobe URL project. Since the strength of sedimentary rock is low and subject to high stress at a depth of 300 m or more where disposal tunnels are to be constructed, it is highly likely that rigid supports will be required. Given that around 300 tunnels (50 tunnels per panel x 6 panels) need to be constructed (JNC, 1999), it would be desirable from an economic viewpoint to reduce the thickness of each concrete support as much as possible. The concept of double support is expected to achieve tunnel stability with the lowest possible support thickness. This concept was used, for example, in the Iiyama tunnel (Kitagawa et al., 2003), famous for swelling ground in fractured zones. In the double support concept, the second support is constructed after the rock has deformed to some extent, subsequent to the installation of the first support. The stress on the whole support will be reduced in the final structure (Matsui et al., 2005a). As described later in section 2.2.3

(3) on important points for repository design, however, use of the double support concept for repository design would require additional studies on sealing performance in the direction of tunnel axis, as is required for plugs.

4) Determination of parameter values for analyses of concrete support

Low pH concrete is now under development and is expected to contribute to reducing the influence of support materials on the engineered barriers and surrounding rock. Details of the development will be described later in section 2.2.6. In this study, type HFSC424 was used as a low pH concrete; it is currently considered suitable for shotcrete because of its strength. Based on the data obtained from laboratory tests, tentative values for the shotcrete (Poisson’s ratio, design strength and equivalent elastic coefficient) were determined for the analyses. At the same time, the properties of HFSC424 and restrictions on design associated with the use of the concrete were clarified (Matsui et al., 2005a). However, this type of concrete is still under development and the results described below will need to be updated in accordance with progress.

The Poisson’s ratio of HFSC424 was determined as 0.2. More data will be required to verify this value. A study made by the Japan Society of Civil Engineers (2002) recommends using the formula "expected compressive strength/ overdesign factor ≥ design-base strength" to check the compressive strength obtained from laboratory tests in order to take into account variability of properties expected for the on-site concrete. Table 2.2.3-4 shows the design-base strength of HFSC424 shotcrete determined using this method (Japan Society of Civil Engineers, 2002), using data from tests conducted by Iriya et al. (2003a).The “core” indicated in Table 2.2.3-4 means the cores sampled from the concrete blown onto the mock-up tunnel and “base” means the concrete placed in a mold in the laboratory after mixing and before transport to the site. Since only limited data are available here, 30 MPa, the value for the base, was conservatively selected for analyses. In actual cases, the design-base strength should be determined based on data for bases and cores cured under site conditions. Such data will have to be collected for the Horonobe URL project to verify and update the determined values. Regarding the elastic coefficient, it is often the case that an equivalent value, in which creep strains and drying shrinkage strain is included, is used as the design value for the shotcrete. The equivalent elastic coefficient is usually determined based on laboratory tests, but there were insufficient data available on low pH concrete for this purpose. Therefore, based on a study by Iriya and Mihara (2003), low pH concrete using HFSC424 and OPC concrete were compared in terms of elastic coefficient, drying shrinkage strain and creep strain at the age of 3 days. Based on the result, it can be concluded that the same relationship between equivalent elastic coefficient and strength as in the OPC concrete should be applied to HFSC424 concrete. The value was therefore determined to be 3,400 MPa, as shown in Table 2.2.3-4 (Matsui et al., 2005a).

For the shotcrete used in underground structures such as tunnels, there is no safety factor specified by the Japan Society of Civil Engineers (1996), whereas the Japan Railway Construction Public Corporation (1996) suggests a safety factor of 2, which, however, may be considered valid only for the case with a secondary lining. For application to other structures in which no secondary lining is planned, such as the underground facilities at Horonobe, some margin should be included. The safety factor was therefore set to 4, the value specified for non-reinforced concrete by the Japan Society of Civil Engineers (2002).

Table 2.2.3-4 Analytical parameter of the low-alkaline cement (HFSC424)

	poison's ratio	reduced elastic modulus	classification	test number	28-day strength (average)	standerd deviation	coefficient of variation	overdesign factor	specified concrete strength
HFSC424	0.2	3,400MPa	base	6	32.7MPa	1.18	3.6%	1.07	30.5MPa
			core	6	39.8MPa	1.75	4.4%	1.09	36.5MPa

(2) Case study design for a disposal facility in a geological environment equivalent to that at the Horonobe 450 m level

For the geological environment at the Horonobe 450 m level (mechanical properties (Table 2.2.1-3), initial stress (Table 2.2.1-4) and thermal properties (Table 2.2.1-5)), a case study design was conducted for a single disposal tunnel and single disposal pit, based on vertical emplacement using low pH concrete supports. The tunnel spacing and the waste package emplacement pitch were also analyzed (Matsui et al., 2005a). The shape and dimensions of the cross-sections used in this case study are shown in Figure 2-1.

1) Case study design for a single tunnel and single disposal pit

The preliminary design for a single tunnel and single disposal pit (determination of support thickness) was conducted by an elastic-perfectly plastic analysis according to the Mohr-Coulomb fracture criterion, based on the design procedures and stability indices (Table 2.2.3-1) described in section 2.2.3 (1)1) (Matsui et al., 2005a). The additional design items for sedimentary rock formations shown in Table 2.2.3-3 were not included in this study and were left as tasks for future studies. Although a three-dimensional model may be preferable for

application to the whole study, a two-dimensional FEM elasto-plastic analysis was used to design the support for the disposal tunnel in this study and the three-dimensional FEM elasto-plastic analysis was used only to evaluate the stability of the disposal pit (Matsui et al., 2005a; Yoshino et al., 2005). The tunnel axis was in the direction of the principal stress (E-W direction), allowing better tunnel stability, and full-face excavation was used. The stress release rate was set at 65% in the two-dimensional elasto-plastic analysis. The three-dimensional elasto-plastic analysis was conducted on the assumption that the length of an excavation cycle would be 1 m and the disposal tunnel would be excavated first, followed by the disposal pit. It should be noted that this case study design is not part of the design of the Horonobe URL.
The disposal tunnel was designed, as a first step, with a low pH concrete lining with a thickness of 20 cm without steel supports. If this condition proved inappropriate, as a second step the thickness of the lining would be increased and the need to use steel supports evaluated. For the disposal pit, it was examined whether the pit would be stable without any support.

The result indicates that, for the geological environment at the Horonobe 450 m level, a lining thickness of 20 cm without steel supports would be appropriate for the disposal tunnel and that the disposal pit would be stable without any support as long as waste packages were emplaced immediately after excavation (no effect of rock creep) (Matsui et al., 2005a). It should be noted that this result is based on the assumption that the rocks at depth of 450 m would be intact without any fractures (see section 3.2). Considering that the design of the Horonobe URL employs steel supports and that the actual geological environment often proves to be heterogeneous, the result of this evaluation should be verified during investigations to be conducted during the construction of the underground facilities. When such an analysis is used in the design, some margin should be included in the values used in the analyses.

2) Study on tunnel spacing and waste package emplacement pitch

Based on the specifications for a single tunnel and single disposal pit defined in the case study design above and based on the evaluation indices shown in Table 2.2.3-2, disposal tunnel spacing and waste package emplacement pitch were determined from mechanical and thermal viewpoints (Matsui et al., 2005a). The reference case in this study uses a disposal tunnel with 2.6 D (where D is the tunnel diameter) and waste package emplacement pitch of 3.0 d (where d is the disposal pit diameter) for vertical emplacement.

(a) Study on disposal tunnel spacing and waste package emplacement pitch from a mechanical viewpoint

The procedure, method and conditions for the analysis were the same as those used for the single tunnel and single disposal pit, except that more than one tunnel is excavated. In the two-dimensional analysis made for the disposal tunnel, two tunnels were excavated in series to determine the influence of the excavation. In the three-dimensional analysis for the disposal pits, the disposal pit in the center of the three disposal pits was evaluated, when they are excavated in sequence after the tunnel has been excavated. The result indicates that a spacing of 2.6 D could ensure tunnel stability with the same support specifications defined for the single tunnel and disposal pit, and that it could be even reduced to 2.0 D. For the waste package emplacement pitch, 3.0 d proved to be adequate to ensure stability. As described in section 2.2.3 (1) 2), however, the design and construction of closely connected tunnels on such a large scale, with 50 tunnels in a single panel, are specific to geological disposal and there is no evidence of such cases in conventional underground facilities, meaning that the design has not been verified. Therefore, a certain safety margin should be included in the evaluation and design. In addition, the design methods and assessment indices for the closely connected tunnels still need to be verified, considering the creep behavior of rock, the influence of hydrostatic pressure deep underground and type of fracturing.

(b) Study on disposal tunnel spacing and waste package emplacement pitch from a thermal viewpoint

Temperature changes with time in the engineered barriers and the surrounding rock due to the heat generated by HLW glass were analyzed to determine values for disposal tunnel spacing and waste package emplacement pitch, to ensure that the temperature of the buffer would remain below the upper limit required to ensure sufficient performance of the engineered barriers (Matsui et al., 2005a). The general-purpose finite element analysis code FINAS (PNC, 1992) was used for this analysis. The composition of the backfill material was revised as described in section 2.2.5 (1) (bentonite 40% + excavated rock 60%), considering that the geological environment at Horonobe (saline groundwater) and the thermal properties of the revised composition were determined using the relationship derived from tests on thermal properties (Matsui et al., 2005a). The result of the analysis indicates that a tunnel spacing of 2.6 D and a waste package emplacement pitch of 3.0 d will keep the temperature in the near-field below the specified limit (HLW glass: 400 – 500°C, buffer: 100°C, rock: 150 – 300°C). It also indicates that the disposal tunnel spacing might be further reduced (Matsui et al., 2005a).

(3) Points to note in designing a repository

Borehole investigations during the surface-based investigation phase provide only a limited understanding of the variety of geological conditions deep underground where disposal tunnels would be excavated and understanding of the geological environment should be improved step-by-step. The surface-based investigations should aim at acquiring data for determining whether formations suitable for constructing the repository can be found at a depth of 300 m or more, and for the detailed design of an access tunnel (shaft) that would be the first step in the construction phase. To achieve the former, it would be necessary to characterize the geological environment with some margin added to the data from the borehole investigations, considering the uncertainties expected in deep underground conditions. Intelligent construction would be important, in which details of the support design or construction method would be constantly updated based on data obtained during actual construction work, as is generally done in other underground construction projects. It is therefore necessary to focus the design analysis on parameters (e.g. critical strain) that can be directly compared with the data (e.g. convergence) obtained during construction and operation.

Specific construction procedures have already been taken into consideration in the detailed design of the Horonobe URL. However, the design method for the Horonobe URL cannot be applied in its entirety to the design of a repository to be constructed in sedimentary rock. Analysis of tunnels and tunnel spacing considering the safety performance of the repository was outside the scope of the design of the Horonobe URL and would have to be reflected in the site selection and in determining disposal depth, as well as in the design and construction. Table 2.2.3-5 shows the specific issues to be addressed in the detailed design of the repository. These issues have not yet been studied and are therefore tasks for future studies.

Table2.2.3-5 Assignments that should be examined for detailed design of the disposal facility that considered safety ability

1	Construction of thinking method of design to EDZ that considered point of view of safe evaluation
2	Chemistry influence of concrete supports and choice of support materials (including support studies of disposal pit)
3	Studies about design and construction method that considered hydraulic pressure of underground deep part

In sedimentary rock with low strength, the rigidity or permeability of the rock around the tunnels may change due to excavation and long-term creep (generation of an EDZ). It is important to take these factors into account in the design or in selection of the site and depth for the repository that has to have a long-term containment capability. However, it is not possible to exactly estimate the potential EDZ or changes in permeability in the surface-based investigation phase. Design and construction methods will therefore have to be developed that can restrict the extent of the EDZ; the self-sealing properties of the rock (Okubo, 2005) will have to be studied and sealing plugs developed in order to ensure the containment function of the repository. As one of those measures, an approach to designing support is described below. There are two types of approach:

(a) Rigid supports that would work against the stress to be installed immediately after excavation so that the loosening process of rocks would be minimized.
(b) Deformation (loosening) is allowed to a certain extent and supports should only sustain the rock where a large stress component is applied (e.g. double-support concept).

A better containment capability will be achieved by approach (a), but this could result in an impractical support thickness and may therefore not be practicable depending on geological conditions. Although the approach in (b) may be preferable from a design viewpoint, it can generate an unacceptable EDZ under certain geological conditions and, in this case, the required containment performance may not be realized. Design approaches and measures to minimize the EDZ should therefore be studied in detail from both safety and construction viewpoints before initiating the actual process of selecting disposal sites and depths. Approach (b) was adopted in the design of the Horonobe URL.

A report by NUMO (2004b) described some requirements for support as shown in Table 2.2.3-6.

Table 2.2.3-6 Design requirements of tunnel support
(Partial revision of the reference [NUMO, 2004b])

requirement of design	Outline
Mechanical stability	・completely maintain hostrock in periods from construction to closure
Chemical stability	・evade that buffer material and hostrock deteriorate ahead of maximum permissible level by contact of concrete-buffer material ,concrete-backfill material and concrete-hostrock ・evade that shotcrete ahead of maximum permissible level by deterioration of concrete material
Construction	・Choice of the support that expression of initial strength is early

Materials for support measures in the repository should be selected such that chemical stability would be ensured as stated in Table 2.2.3-6 (avoiding excessive deterioration of rock, buffer or backfill due to contact with concrete). The H12 Report and the study by NUMO (2004b) employ, in principle, disposal pits without support considering chemical stability requirements. However, in the sedimentary rock at the Horonobe URL site (with unconfined compressive strength of 5-20 MPa), even at the 450 m depth level where a relatively high unconfined compressive strength of 16.3 MPa is assumed, disposal pits would not be self-supporting after several years due to reduced rigidity of the surrounding rock, although they would remain stable for a short period of time (Ishijima and Kiyama, 2004; Yoshino et al., 2005). Therefore, in order to secure tunnel stability for the period from excavation to emplacement in sedimentary rock, measures including providing support would need to be applied as necessary.

It is pointed out that highly alkaline solutions cause the dissolution of smectite, the main mineral of the clay buffer, and would have an impact on the performance of the buffer (Shibata et al., 2004).

Regarding the low pH cement being developed, a seawater exposure test with HFSC226 showed earlier corrosion of the rebar than in the case of OPC, as described in section 2.2.6. It was therefore recommended that, with the current status of development, HFSC should not be used together with ordinary rebars. Further studies should be carried out to evaluate the applicability of OPC and low pH concretes in the repository and to identify restrictions associated with their use.

A considerable hydrostatic pressure will be applied to deep underground structures. Measures in the design and construction aimed at addressing this pressure include reducing the water pressure applied to the supports by providing a conduit, as in the Horonobe URL, reducing the pressure by ground improvement using grouting or freezing methods, providing resistance to the pressure by installing segments or other rigid supports. For the disposal project, however, the most appropriate approach would be not to leave any water channels on closure of the repository, because consideration should be given not only to the construction feasibility but also the confinement capability of the design. Further studies should be carried out in this respect.

(4) Other studies

Among the other items studied in the course of the Horonobe URL design, the following are items that may be relevant to the design of a repository in sedimentary rock.

1) Study on seismic resistance

It is known from experience that underground structures such as tunnels generally suffer relatively little impact from earthquakes and past studies also show that the seismic ground motion at depth is several times lower than that measured at the surface. However, a geological repository is a large-scale project with shafts (access tunnel) and horizontal drifts to be constructed to and at depths of 300 m or more; there is no precedent for this in domestic construction history. From a safety assessment perspective, it is required that the geological environment around the repository and tunnel stability will not be affected by earthquakes. Tunnel stability in the event of earthquake should therefore be checked for the geological environment of interest. In the Horonobe URL project, seismic resistance was evaluated for the shaft, horizontal drift and intersections between the tunnels. It was confirmed that no revision in the design of the support is required to prevent against potential earthquake impacts in general. The only exception was that concrete with a higher design strength should be used for a limited part of the shaft support located in the weak layer extending to a depth of several dozen meters from ground surface (Yuchi Formation) (Matsui et al., 2005a). In the evaluation of seismic resistance for the Horonobe URL, the input seismic wave was formulated based on investigations of past earthquakes and active faults in the area around the URL and a mechanical analysis was made of the response of the shaft and surrounding rock when this seismic wave was applied. The applicability of the static analysis method for the integrity evaluation was also tested by converting the seismic force to equivalent seismic intensity (mechanical shear stress was converted to static seismic intensity). The stress state of the concrete lining around the shaft and that of the surrounding rock was analyzed against the applied equivalent seismic intensity using the static analysis method and compared with individual allowable stress to check integrity. Details on the method used in the seismic resistance study as part of the design of the Horonobe URL are provided in the report "Geoscientific research" (Matsui et al., 2005a).

2) Study on methane gas blowout

One of the features of the geological environment at Horonobe is that it bears methane gas. In geological environments that bear methane gas, a serious accident may occur as a result of the methane gas filling the tunnels. Therefore, it is necessary to estimate the methane gas blowout accurately, in order to plan appropriate measures such as ventilation control to keep the concentration of methane under specified limits. In the Horonobe URL, a gas volume estimation was made and the result was reflected in the ventilation design (Matsui et al., 2005a). It will also be necessary for a repository to be constructed at a methane gas-bearing site to make such analyses and include measures against gas in the design, using, for example, the methods documented in the study by Matsui et al. (2005a).

3) Study on technologies reflecting measured data in the design and construction plan (intelligent construction)

Intelligent construction is a method in which in-situ measured data are collected and analyzed during the construction of the underground tunnels and, based on the results, the original design is re-evaluated and, if required, revised and/or additional construction measures are introduced to facilitate construction. The construction of the underground facilities in the Horonobe URL project will be the first case in Japan that applies the intelligent construction approach to large-scale underground facilities in Neogene sedimentary rock and approach has been taken into account in the design process (Matsui et al., 2005a).

Figure 2.2.3-2 shows the process flow for intelligent construction during construction phases. The measurements, which form the basis of the entire process, consist of both daily and stepwise measurements. The former are basically carried out for every excavation unit (1.3m length). The measurement results, including geological observations, are used mainly for determining support specifications and construction methods and are reflected directly in the next step. This provides an understanding of the state and behavior of the rock around the tunnels and allows evaluation and revision of the analytical models used in the design process. The support specifications and safety standards are also evaluated and revised accordingly for the excavation process in the next phase.

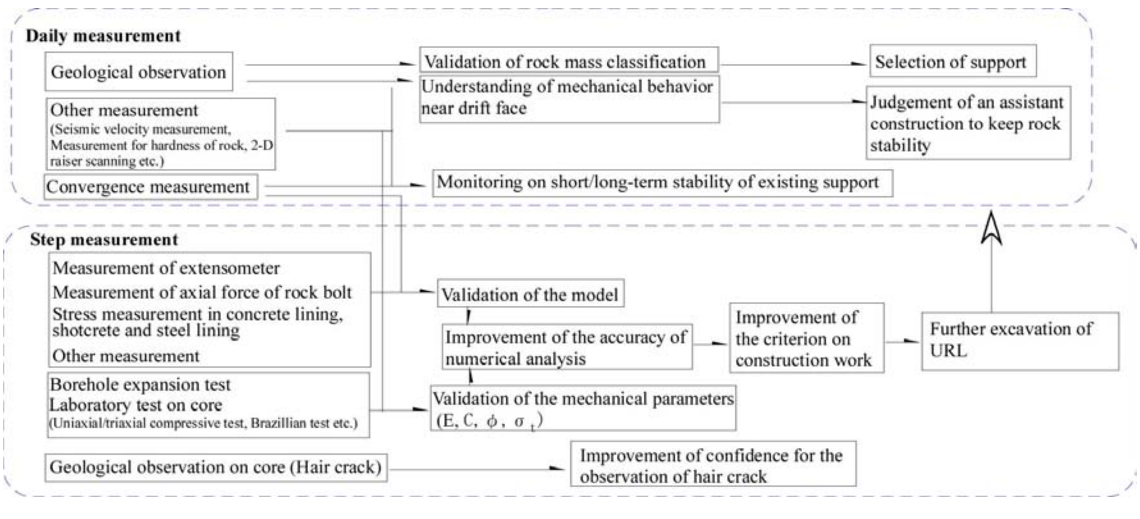


Figure 2.2.3-2 Intelligent construction system flow
on the construction of Horonobe URL facility

2.2.4 Applicability study of the EBS design method

The design flow for the buffer and overpack was established based on the relationships between the overpack, buffer and backfill materials in the design process as described in section 2.1.2, and also on the design flow for the whole repository system revised based on the study of these relationships. The results are outlined below.

(1) Concept for the design procedures

1) Concept for the buffer design procedure

The design of the engineered barriers will follow the design flow shown in Figure 2.2.4-1. Some of the design requirements for the buffer do not require information from the overpack design. In this study, therefore, design requirements for the buffer were divided into two categories: those which require information on overpack design and those which do not. The tentative specifications of the buffer were defined based on the latter, which are then reviewed from the perspective of other requirements after the overpack specifications have been defined (Matsui et al., 2005a).

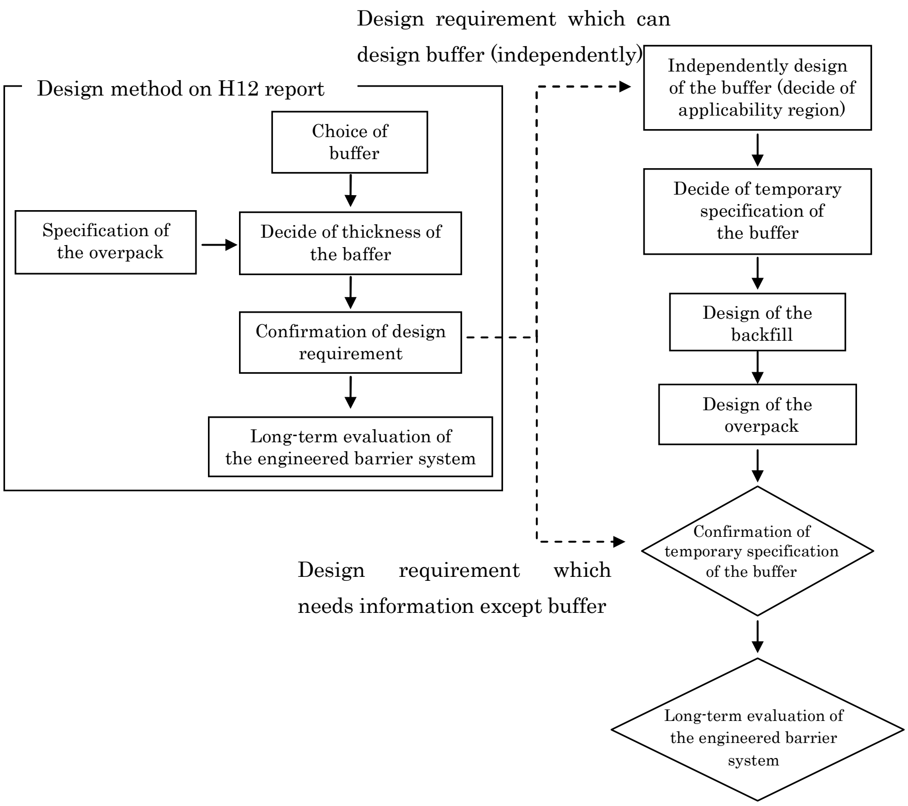


Figure 2.2.4-1 Comparison with a design procedure on the H12 report

The procedure used in this study, included the following, is shown in Figure 2.2.4-1 (Matsui et al., 2005a)

(a) Design considering, inter alia, the low permeability, self-sealing and colloid filtration properties, as well as manufacturability, to determine the range of specifications that satisfy these requirements.
(b) Defining the tentative specifications within the range defined in step (a).
(c) After the specifications for the overpack and backfill have been defined, the validity of the tentative specifications defined in step (b) is checked in terms of stress buffering capability, support of the vitrified waste and self-sealing properties.

In step (a), the range of effective clay density will be determined. Regarding the self-sealing properties, the lower limit should be defined based on the volume of the gap around the buffer, which should, in turn, be calculated based on the dimensions of the buffer and overpack. Therefore, in step (a), the lower limit was determined on the assumption that the buffer would fill the gap, regardless of the gap volume, when the material has reached a swelling pressure of 0.1 MPa (Dixon, 2000) after self-sealing of the gap around the buffer. Whether the defined value is appropriate for the gap to be expected during actual operations will be evaluated in step (c). An example of the procedure for designing the buffer is shown in Figure 2.2.4-2, in which the specification range is given for all four aspects: low permeability, self-sealing, colloid filtration properties and workability.

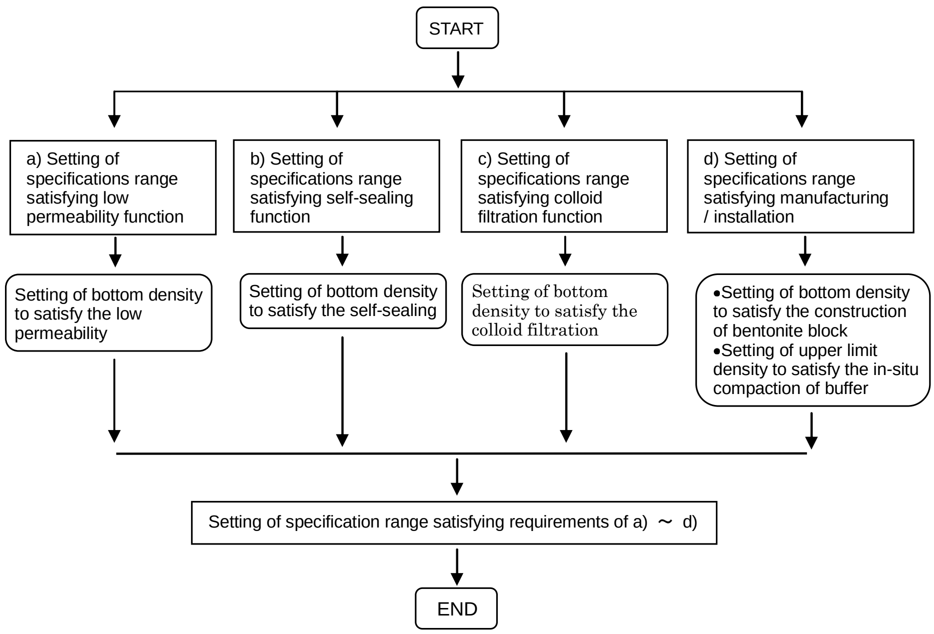


Figure 2.2.4-2 Example of design procedure for buffer material

2) Concept for the overpack design procedure

The main points to be considered in designing the overpack for the geological environment at Horonobe are the influence of groundwater chemistry (saline water) and rock behavior on corrosion and pressure resistance. These points are especially important for determining the thickness of the overpack. Figure 2.2.4-3 shows design procedures using information on the geological environment for candidate materials carbon steel, titanium and copper (Kurihara et al., revised in 2004). Feedback routes are ignored in this design flow diagram. The design of the overpack was conducted according to this flow focusing on thickness. The major points featured in the design are shown framed red in Figure 2.2.4-3 and are described below.

- Corrosion lifetime: The corrosion behavior of carbon steel, titanium and copper are evaluated for porewater with the chemistry of the groundwater sampled in the Horonobe URL project (Horonobe groundwater). The result is used to determine the overpack corrosion allowance. An important point is to judge whether the Horonobe groundwater has any influence on the corrosion behavior of the different candidate materials.

- Corrosion allowance: The amount of corrosion is calculated based on the volume of oxygen introduced around the overpack for corrosion due to oxidation. For corrosion due to reduction of water, it is calculated based on the corrosion rate in a reducing environment and considering an overpack lifetime of 1,000 years.

- Thickness for pressure resistance: The thickness to provide the required pressure resistance (hydrostatic pressure plus the consolidation reaction force of the buffer at the target depth) is determined by strength evaluation. It is important to determine the hydrostatic pressure and the consolidation reaction force of the buffer depending on rock behavior and groundwater chemistry in the Horonobe environment.

- Thickness for radiation shielding: The thickness required for radiation shielding is determined by converting the flux of oxidizing chemical species produced by water radiolysis to cathodic current density (JNC, 1999).

- Workability: The feasibility of the overpack thickness and specifications determined as above are evaluated from manufacturing and construction viewpoints.

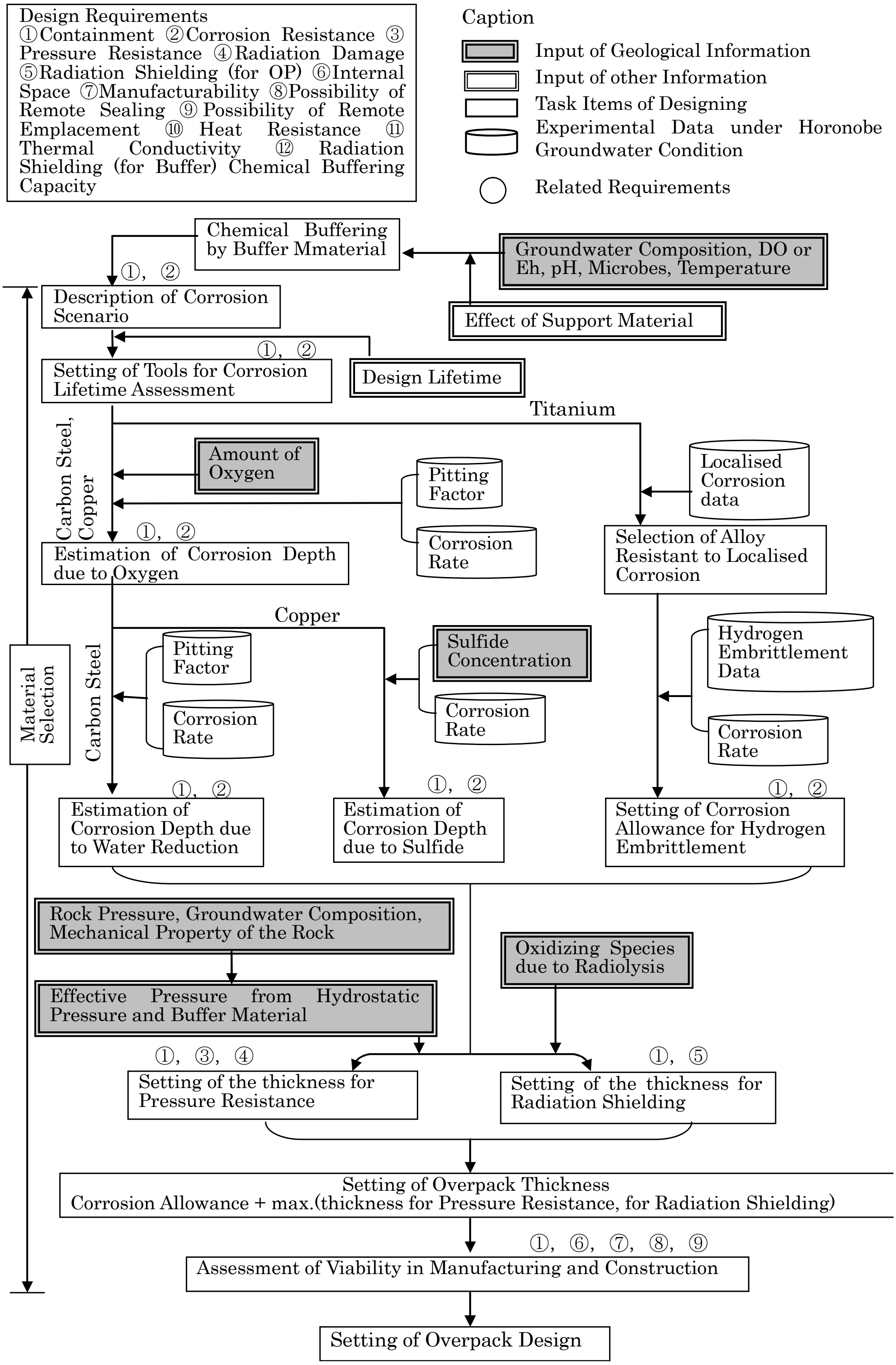


Figure 2.2.4-3 Design Flow of Overpack (Revision of Kurihara et al., 2004)

3) Evaluation of long-term integrity of the engineered barrier system

The long-term behavior of the engineered barrier system consisting of the buffer and overpack with the specifications defined above was analyzed to evaluate system integrity. Figure 2.2.4-4 shows the analysis flow (Matsui et al., 2005a). The analyses were conducted individually for each of four aspects: mechanical deformation of rock, mechanical deformation of the buffer, gas migration through the buffer and extrusion and erosion of the buffer into the fractures, and four analyses are therefore shown in parallel in the figure. If the results show that the determined specifications of the overpack and buffer cannot satisfy the required long-term integrity for one or more of these aspects, they should be revised according to the design flow shown in Figure 2.1.1-2.

(2) Design of the buffer

A case study design of the buffer was conducted for the geological environment conditions at the Horonobe 450 m level according to the procedure described above (Matsui et al., 2005a). The influence of saline groundwater was also taken into consideration since it was reported that the permeability and swelling pressure of the buffer tend to be lower under these conditions compared to a freshwater environment (Kikuchi et al., 2004).

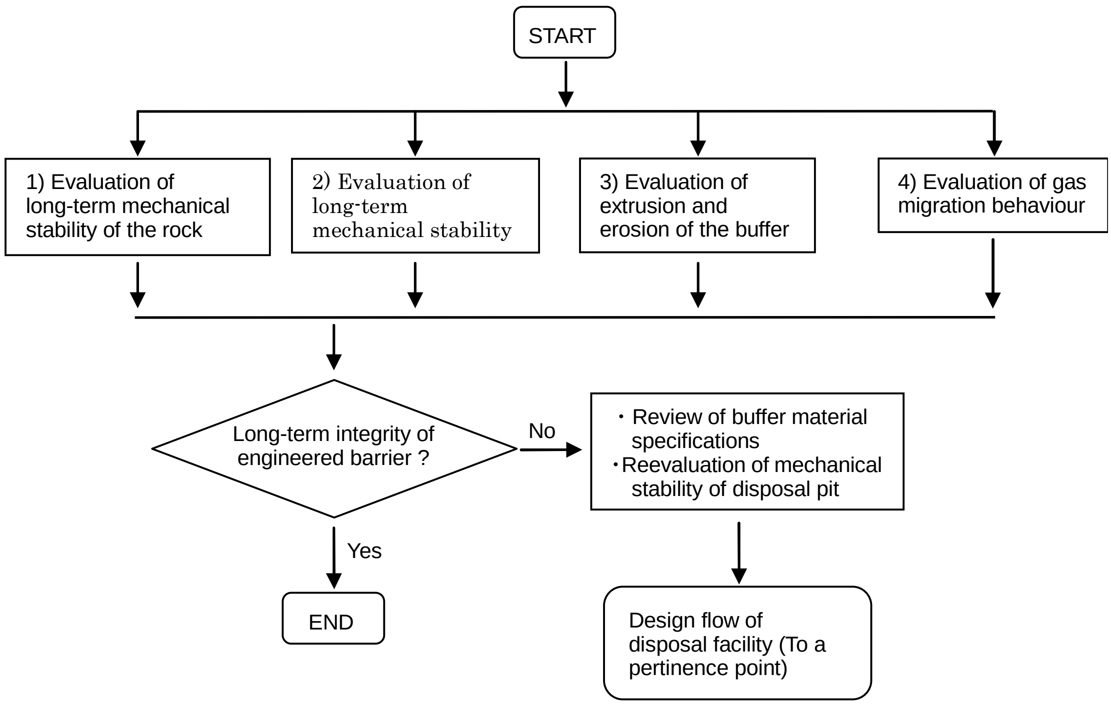


Figure 2.2.4-4 Evaluation flow of the integrity of the engineered barrier system

1) Determination of the specification range and tentative specifications of the butter material

With regard to the function of restricting groundwater movement, the Peclet number was evaluated based on the hydraulic gradient at Horonobe described in section 2.2.1 (2) 5) to determine the lower limit of density at which the buffer still maintains a diffusion-dominated field (Matsui et al., 2005a). For colloid filtration, an effective clay density of 0.8 Mg m^{-3} or more was used as the lower limit, which is the same value as for freshwater conditions since there are no experimental data for saline groundwater conditions (Matsui et al., 2005a). Regarding the self-sealing properties of the buffer, a swelling pressure of 0.1 MPa or more was used as the criterion for the self-sealing property of the buffer, which is employed as one of criteria with POSIVA (Finland). The lower density limit was then determined considering the saline groundwater conditions at Horonobe (Dixon, 2000). For manufacturing and construction workability, an upper density limit was determined for the emplacement methods in-situ compaction and block system (Matsui et al., 2005a). Figure 2.2.4-5 shows an example of the specification range determined for the block system. The specifications defined in the H12 Report (dry density of 1.60 Mg m^{-3}, sand mixing ratio of 30 wt% and effective clay density of 1.37 Mg m^{-3}) are also shown in the figure. The figure indicates that the dry density of 1.60 Mg m^{-3} and the sand mixing ratio of 30 wt% are appropriate even for the geological environment conditions at Horonobe. These values were used for the tentative specifications.

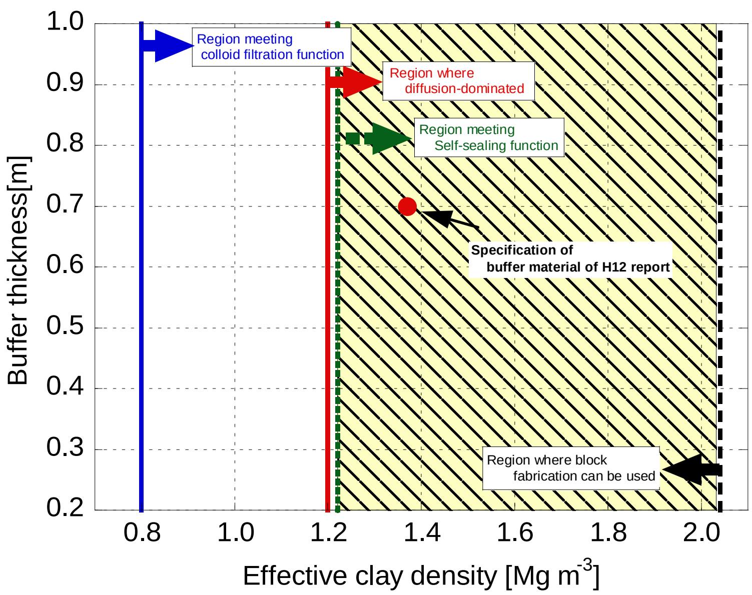


Figure 2.2.4-5 Estimation of temporary specification of the buffer for example block installation method

2) Evaluation of tentative specifications of the buffer

With regard to self-sealing, the width of the gap generated between the buffer and the overpack or the surrounding rock when the HLW glass is emplaced was assumed to be as shown in Table 2.2.4-1 for the in-situ compaction method and block system. The gap volume was calculated accordingly and an evaluation was made using data obtained from a gap swelling test (Kikuchi and Tanai, 2005). It was confirmed that the tentative specifications would fill the gap sufficiently (Matsui et al., 2005a). The ability to support the overpack was evaluated using the tentative specifications of the buffer and the calculation method for vertical load-bearing capacity specified in the Building Foundations Structure Design Guideline (Architectural Institute of Japan, 2001) to be on the conservative side. The calculated value was then assessed based on the assumption that, if the unconfined compressive strength of the buffer placed under the overpack was higher than the load on the buffer, it could support the overpack. The validity of the tentative specifications was therefore confirmed for a sufficient overpack supporting capability (Matsui et al., 2005a). The stress buffering capacity was evaluated taking the groundwater chemistry, underground pressure, hydrostatic pressure and mechanical properties of the rocks at Horonobe into account and the validity of the tentative specifications was confirmed (Sugino et al., 1999; Matsui et al., 2005a). Taniguchi et al. (2002) reported an experimental relationship between the passivation of carbon steel and dry density and sand mixing ratio of the buffer. The tentative specifications were therefore evaluated for the influence on passivation of carbon steel (material for the overpack) and it was confirmed that these specifications are in the range that would not cause passivation of carbon steel as shown in Figure 2.2.4-6 (Matsui et al., 2005a). It should be noted that the groundwater used in the experiments in Figure 2.2.4-6 had a higher carbonate concentration and did not contain chloride ions, compared with the groundwater chemistry at Horonobe, indicating conditions that are more likely to promote passivation of carbon steel. Figure 2.2.4-7 summarizes the results of an independent specification study, as well as an evaluation of the tentative specifications obtained. As seen in the figure, it was confirmed that the tentative specifications defined in this case study design (dry density of 1.60 Mg m^{-3} and sand mixing ratio of 30 wt%) could achieve the required performance under the geological environment conditions at the Horonobe 450 m level.

Table 2.2.4-1 Temporary specifications of a gap（JNC，1999）

Block installation method	A gap to occur between the buffer and the rock mass	40mm
	A gap to occur between the buffer and the waste package	20mm
In-situ compaction method	A gap to occur by limitation of handling (between the buffer and the waste package)	20mm

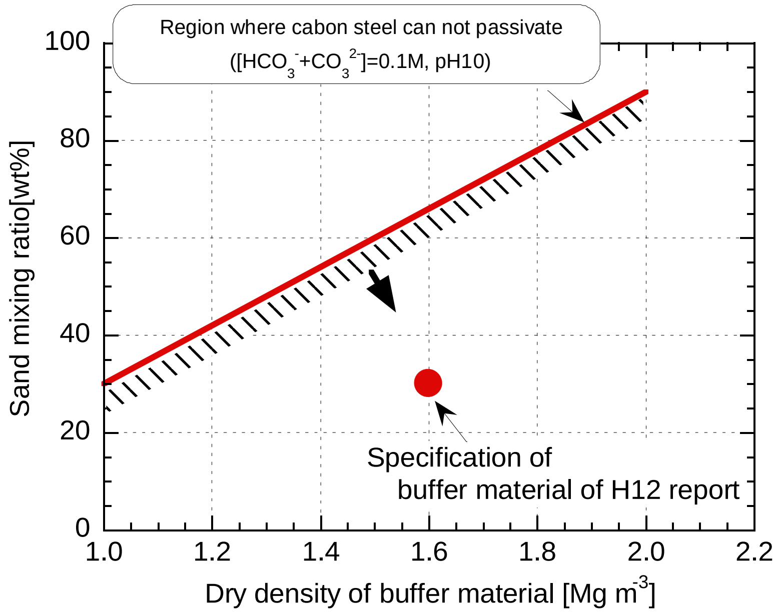


Figure 2.2.4-6 Relationship between passivation condition of carbon steel overpack and specification of the buffer (dry density and sand mixtures ratioto)

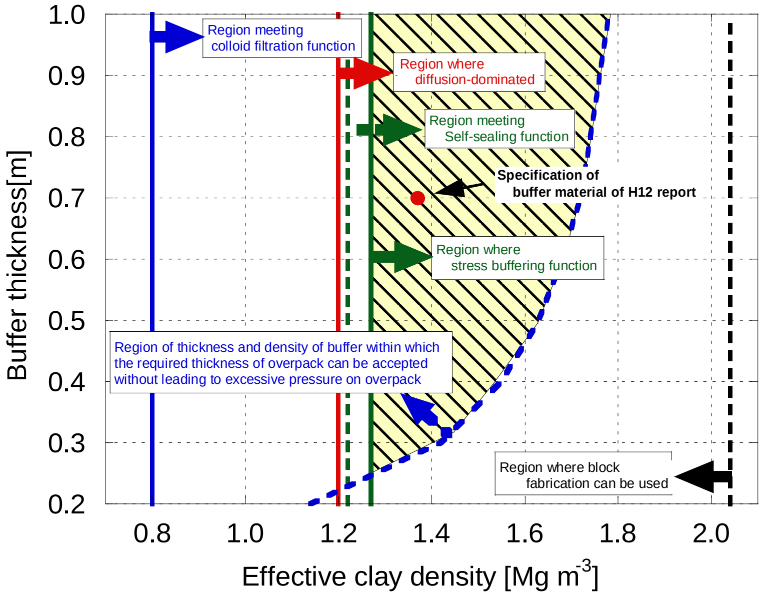


Figure 2.2.4-7 Evaluation of specification of the buffer
for example block installation method

(3) Design of the overpack

Specifications for the overpack were determined according to the design flow, considering the information on the geological environment as described above and using the specifications defined separately for the buffer and backfill.

The thickness of the overpack was determined to be 190 mm, based on an evaluation of corrosion resistance for the Horonobe groundwater chemistry, the specifications of the buffer and backfill, the consolidation reaction force of the buffer and hydrostatic pressure (Matsui et al., 2005a). A summary is given below.

1) Corrosion resistance

Carbon steel is considered not to be passivated and is therefore subject to general corrosion under the groundwater conditions at Horonobe (saline water, carbonate concentration of 0.1 mol l^{-1} or less, pH value in the range 6 to 7, chloride ion concentration in the range of approximately 0.1 to 0.4 mol l^{-1} (Taniguchi et al., 2002).

For the titanium, crevice corrosion was reviewed as this is the type most often observed in titanium. Pure titanium may be subject to crevice corrosion at a chloride ion concentration of 10^{-3} mol l^{-1} or higher, which would also be the case under the Horonobe groundwater conditions. For the titanium alloy, a review was carried out for Ti-Ni-Mo alloy (ASTM Grade 12) and Ti-Pd alloy (ASTM Grade 7, 17, etc.) (Nakayama et al., 2002). It is reported that the former is expected to be sensitive to crevice corrosion but the latter is not, and the same trend is expected for the groundwater conditions at Horonobe. Even in the case where no crevice corrosion results, the passivation film on the titanium could become unstable under certain environmental conditions. However, the conditions at Horonobe are outside this range and the corrosion resistance would not be significantly degraded.

With regard to the copper, electrochemical tests in the past have shown that the copper is barely passivated under a wide range of solution conditions and that almost all polarization curves are classified as active dissolution type (Kawasaki et al., 2002), which should certainly be the case under the conditions at Horonobe with a high chloride ion concentration. The copper should therefore have a low risk of undergoing localized corrosion due to local damage of the passivation film.

No negative factors have been found so far for any of three candidate materials to be used for the overpack under the conditions at Horonobe.

2) Determination of overpack thickness

The thickness should be the sum of the corrosion allowance for groundwater corrosion and the thickness that satisfies the requirements for resistance against loading such as the overburden, hydraulic pressure and swelling pressure of the buffer, or that for radiation shielding, whichever is larger. The results of the study are summarized below for the case where carbon steel is used for the overpack with a design lifetime of 1,000 years.

The corrosion allowance was determined by adding the amount of corrosion caused by oxygen and by reduction of water. The amount of corrosion by oxygen was calculated assuming that all the oxygen contained in the buffer and backfill around the overpack would be consumed for the corrosion. The maximum corrosion depth determined in this way was 15 mm. For corrosion due to water reduction, it is reported that the average corrosion rate for a three-year experiment with simulated seawater was in the range 6 to 7 μm y^{-1} (Taniguchi et al., 2004). Accordingly the value was conservatively set to 10 μm y^{-1}, resulting in an average corrosion depth of 10 mm in 1,000 years. The maximum depth was set to twice the average, i.e. 20 mm, considering possible heterogeneity in the density. The corrosion allowance based on the above discussion was 40 mm.

For the pressure resistance, a plate thickness was calculated that would withstand the combined external load from hydrostatic pressure of the groundwater (4.41 MPa) and the consolidation reaction force of the buffer (1.52 MPa) according to the regulations for second-class containers specified in the "Technical Standard for Structures of Nuclear Power Plant Systems" (Notification No. 501 of the Ministry of International Trade and Industry, 1980). The result was 80 mm for the cover and 30 mm for the body.

For the radiation shielding thickness, the radiation field from the HLW glass should be analyzed to determine the absorbed dose on the surface of the overpack. Based on this, the corrosion of the overpack is calculated under the assumption that all the oxidizing chemical species generated by water radiolysis would be supplied to the overpack surface and would contribute to corrosion (JNC, 1999). Since the characteristics of the HLW glass and specifications of the buffer in the H12 Report have not been revised, the thickness required for radiation shielding was also assumed to be the same, namely 150 mm.

From the above discussions, the total thickness was determined to be 190 mm, i.e. the sum of the radiation shielding thickness of 150 mm and the corrosion allowance of 40 mm.

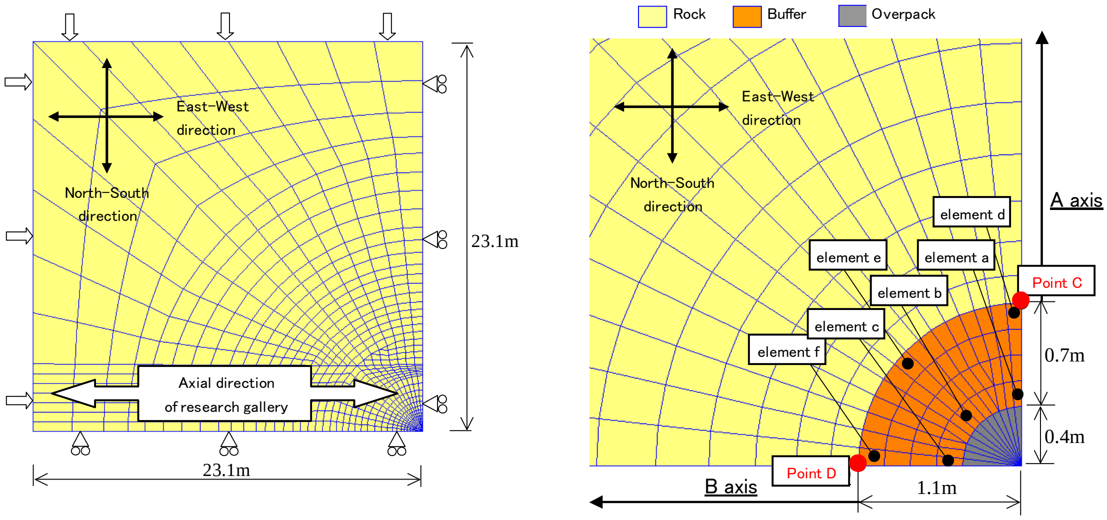
(4) Long-term integrity of the engineered barriers

The long-term integrity and construction feasibility of the engineered barrier system with the specifications determined above were evaluated for a single disposal pit under the geological conditions at the Horonobe 450 m level, in terms of long-term mechanical deformation of the rock and buffer, extrusion and erosion of the buffer into the fractures and gas migration through the buffer. The result is described below.

1) Long-term mechanical deformation of the rock

The long-term integrity of the disposal pit for vertical waste emplacement was evaluated from a mechanical viewpoint. A trial calculation was made of long-term mechanical deformation up to 10,000 years after emplacement. Displacement of the disposal pit wall, the extent of the zone of reduced rigidity around the disposal pit (zone over which the EDZ generated by excavation extends) and the stress of the buffer were evaluated analytically (Matsui et al., 2005a).

The model used in the analysis is shown in Figure 2.2.4-8 (horizontal view of the disposal pit). The study was made based on the assumption that the disposal pit stood alone, i.e. no consideration of neighboring disposal pits, and no support was constructed for the disposal pit. The buffer and overpack were modeled as linear elastic solids. For the most conservative evaluation, the swelling of the buffer and expansion of the overpack due to corrosion were not included so that the influence of creep would become obvious in the result. A valuable-compliance-type constitutive equation that is a visco-elastic model was used to express rock behavior and the Mohr-Coulomb fracture criterion was applied. The properties used in the analysis are shown in Tables 2.2.1-3, 2.2.1-4 and 2.2.4-2. Constants n and m to be used in the a valuable-compliance-type constitutive equation were determined as "n, m = 30, 20" based on laboratory tests using rocks sampled from boreholes HDB-3 and HDB-6 (Ohkubo, 2004, 2005). It was assumed that the maximal horizontal principal stresses (S_{max}) would be in the axis direction of the experiment drift (E-W direction) and that the initial stress ratio would be “vertical: E-W: N-S = 1 : 1.3 : 0.9” (Matsui et al., 2005a).



(a) General view (b) Enlargement around disposal pit

Figure2.2.4-8 2D long-term dynamics analysis model
(a model that looked at disposal pit planarly)

Table2.2.4-2 Dynamics properties of artificial barrier used for calculation

	Buffer	Overpack
Young's modulus [MPa]	18.31	210,000
Poisson's ratio [-]	0.4	0.2

The mechanical properties of a disposal pit in a sedimentary rock environment with low strength and under asymmetric pressure, such as found at Horonobe, should be impacted by the excavation of the disposal tunnels. A three-dimensional elasto-plastic analysis was therefore carried out first and the obtained stress was then used as the initial stress in the two-dimensional long-term mechanical behavior analysis. Figure 2.2.4-9 shows the used in this analysis. For step 3, construction of the engineered barriers was assumed to occur immediately after excavating the disposal pit.

step1 Dead load analysis
（Calculate stress states after excavating test gallery in 3D elasto-plastic analysis）
↓
step2 Excavate disposal pit
（d＝2.2m）
↓
step3 Installation of artificial barrier
（Install immediately after excavating disposal pit）
↓
step4 10,000 years progress

Figure2.2.4-9 Procedure of 2D long-term dynamics analysis

The findings of the analysis of the influence on the engineered barriers and the extent of the zone of reduced rigidity are as follows.

- Displacements of the disposal pit wall are 4.8 mm (point D) and 13.5 mm (point C) for 1,000 years and 5.8 mm (point D) and 17.0 mm (point C) for 10,000 years, as shown in Table 2.2.4-3. The values shown here represent the displacement for a disposal pit radius of 1.1 m.

- The stress in the buffer remains under a critical state approach degree of 1 after 10,000 years, as shown in Figure 2.2.4-10, which means that the buffer will maintain its integrity even after 10,000 years.

- The stress in the rock around the disposal pit (Figure 2.2.4-11) is mainly in the direction of the A axis due to unsymmetric pressure in the rock and the width of the loosened zone should be 0.2 d after one year and 0.5 d after 10,000 years, where d is the diameter of the disposal pit. The weak zone here means a zone in which Young's modulus E has decreased from its initial value, i.e. the zone in which the effect of creep has propagated.

Table2.2.4-3 Quantity of wall surface deformation in each elapsed time

	Quantity of wall surface deformation		
elapsed time [year]	1	1,000	10,000
A-axis direction, point-C [mm]	-8.6	-13.5	-17.0
B-axis direction, point-D [mm]	-2.9	-4.8	-5.8

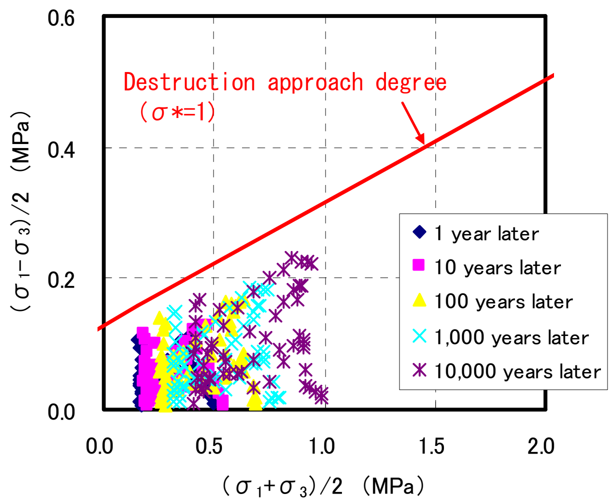
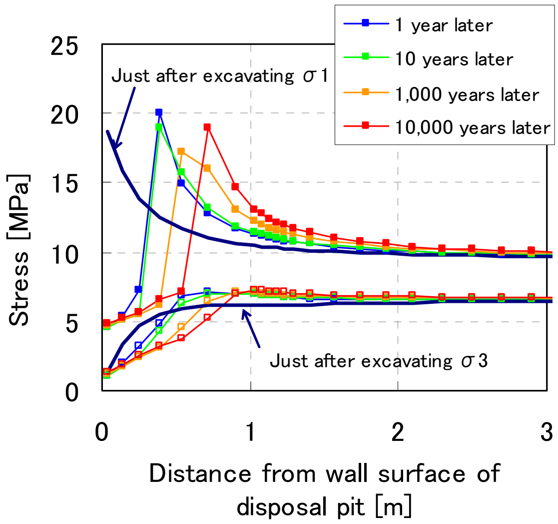
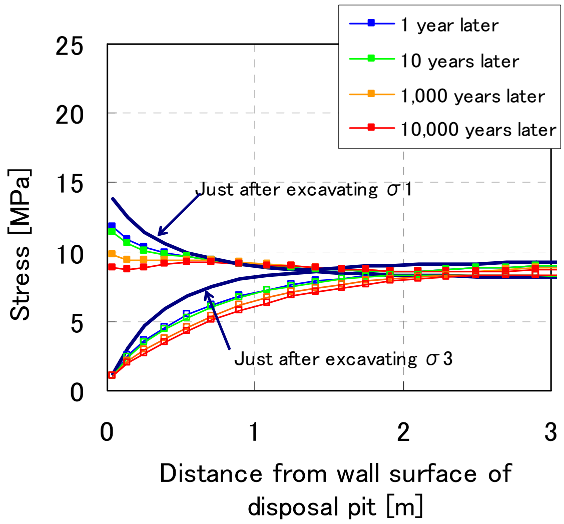


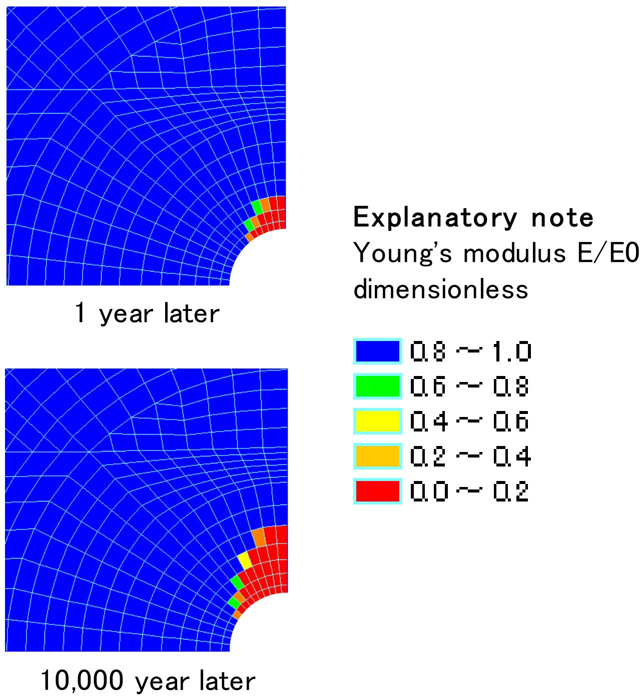
Figure2.2.4-10 Stress state of buffer



(a) A-axis direction



(b) B-axis direction



(c) Distribution of hardness fall domain

Figure2.2.4-11 Stress state of rock around disposal pit

Future tasks with regard to the analyzing the long-term mechanical behavior of the rock are summarized as follows.

- This study was conducted on the assumption that the engineered barriers would be emplaced immediately after excavation, whereas the actual disposal process will require a certain time period between excavation and barrier emplacement. The disposal pit design and long-term integrity analysis should therefore include influence of the time period from excavation to emplacement.

-An analysis of long-term mechanical behavior should be conducted not only for the rock around the disposal pit, but also for the disposal system as a whole, including the disposal tunnels, taking the effects of neighboring tunnels into account.

- Swelling of the buffer or expansion of the overpack due to corrosion may cause tensile fracturing in the rock around the tunnels. An analysis approach that includes these factors should therefore be developed.

2) Evaluation of the long-term mechanical deformation of the buffer

The long-term mechanical deformation of the buffer to be installed in the geological environment at Horonobe was evaluated for buffer material with the defined specifications (bentonite mixed with 30% sand and a dry density of 1.6 Mg m^{-3}). An analysis was made of the influence of sinking due to the deadweight of the overpack and expansion due to corrosion of the overpack, as well as the influence on the colloid filtration and self-sealing functions of the buffer (Matsui et al., 2005a).

The constitutive models of Sekiguchi and Ohta model (Sekiguchi and Ohta, 1977) and Adachi-Oka model (Adachi and Oka, 1982) were used to represent the behavior of the buffer. These models were developed to analyze two fundamental factors in evaluating the long-term mechanical deformation of the buffer: elasto-plasticity (with a non-linear stress-strain relationship) and time dependency (increasing deformation with time under constant stress; creep deformation behavior). Details of both models can found Hirai et al. (2004). The code MuDIAN (Takenaka Corporation, 1993) was used in the analysis.

Table 2.2.4-4 shows the parameters used in the constitutive models; these were determined based on tests using groundwater sampled in borehole HDB-6. Other parameters used for the buffer and overpack are shown in Table 2.2.4-5. The hydraulic conductivity of the buffer was determined using the equation (2.2.4-1) and (2.2.4-2), which were derived from tests using simulated seawater (Kikuchi et al., 2004), considering that the conductivity varies depending on density.

Table 2.2.4-4 Parameters for constitutive model

	Item		Symbol	Unit	Fixed Values
Elasto-plastic property	Compression index		Λ	[-]	0.117
	Swelling index		Κ	[-]	0.043
	Poisson’s ratio		ν	[-]	0.144
	Critical state parameter		M	[-]	0.680
Visco-plastic property	Sekiguchi-Ohta Model	Secondary compression index	α	[-]	1.00×10^{-3}
		Reference volumetric strain rate	$\dot{v}_0$	[h-1]	1.02×10^{-8}
	Adachi-Oka Model	Viscoplastic parameter	m'	[-]	42.5
			C	[h-1]	1.98×10^{-8}

Table 2.2.4-5 Physical properties for overpack and buffer

	Item	Symbol	Unit	Fixed Value
Overpack	Modulus of elasticity	E	[MPa]	2.1×10^5
	Density	ρ	[Mg m^{-3}]	6.63
Buffer	Dry density	ρ_d	[Mg m^{-3}]	1.60
	Coefficient of overburden pressure at rest under pre-consolidation	K_0	[-]	1.00
	Initial coefficient of overburden pressure at rest	K_0	[-]	1.00
	Density	ρ_w	[Mg m^{-3}]	1.00
	Initial total hydraulic head	h	[m]	0.00

$$k = \frac{\rho g}{\mu} K \qquad (2.2.4\text{-}1)$$

$$K = \exp(-47.155 + 15.138\rho_e - 7.878\rho_e^2) \qquad (2.2.4\text{-}2)$$

where k is the hydraulic conductivity [m s^{-1}], K is the intrinsic permeability [m^2] and ρ_e is the effective clay density [Mg m^{-3}].

The analysis model is shown in Figure 2.2.4-12. The constraint conditions for the upper part of the buffer was a free end for the deadweight sinking analysis in order to avoid underestimating overpack sinking; for the corrosion expansion analysis it was a fixed end in order to avoid underestimating the stress generated in the buffer due to overpack corrosion expansion.

The volumetric expansion pattern of the overpack in the corrosion expansion analysis was assumed to be as shown in Figure 2.2.4-13, i.e. the thickness of a hollow cylinder would increase equally at a constant rate in both radial and height directions. The volume of the corrosion product was assumed to be three time larger than that of the original metal. With a corrosion rate of 0.04 mm y^{-1}, it would take 4,750 years to complete corrosion of the entire thickness of 190 mm and the increase in thickness would amount to 184.8 mm in this period of time.

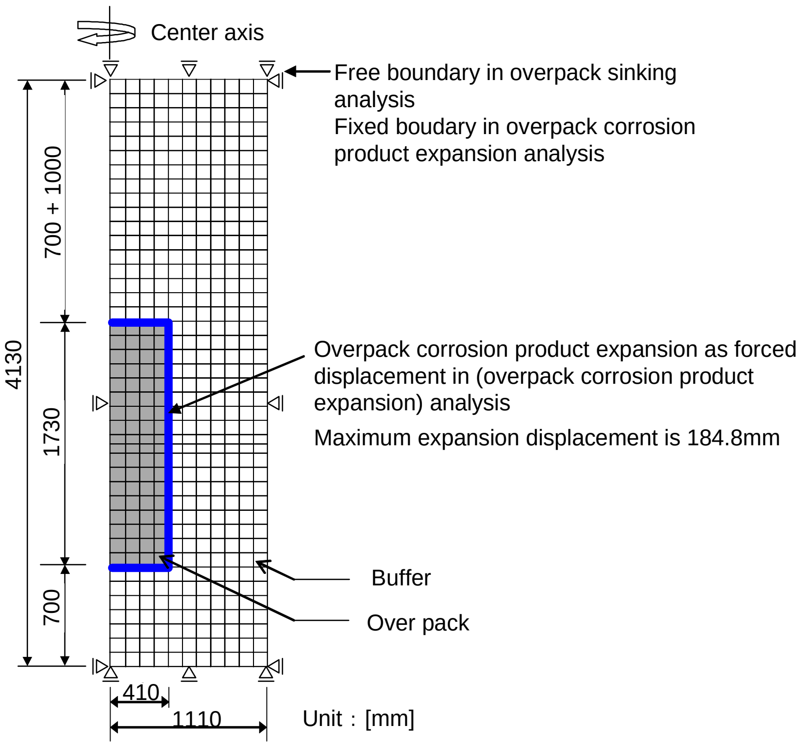


Figure 2.2.4-12 Analytical model of the long-term mechanical deformation of the buffer

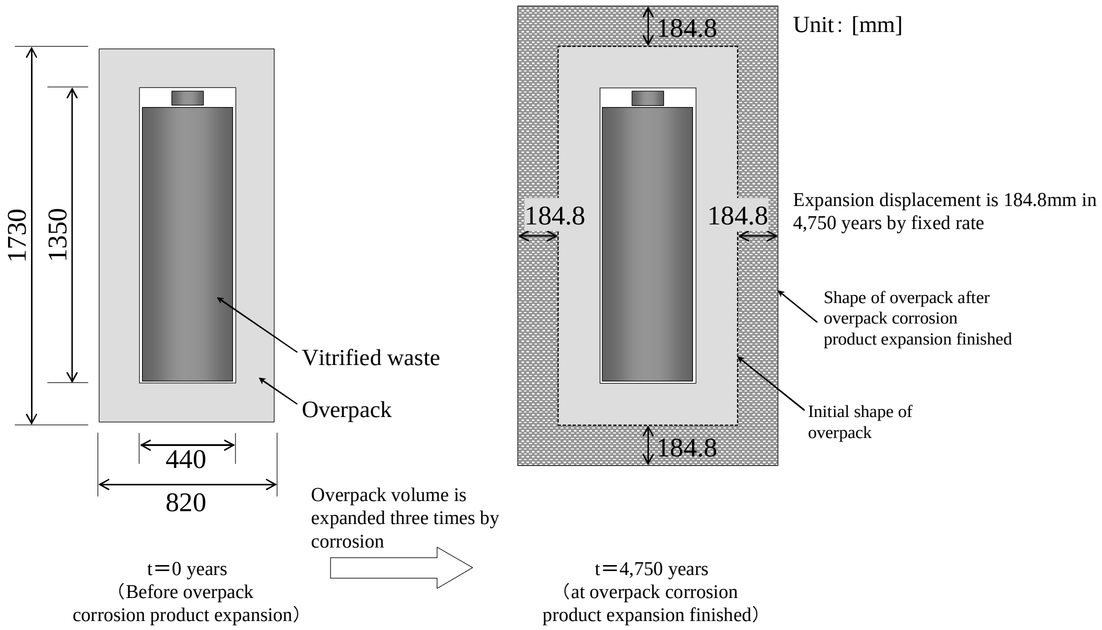


Figure 2.2.4-13 Deformation of overpack due to expansion of corrosion products

Figure 2.2.4-14 shows displacement with time due to sinking in the deadweight sinking analysis. The displacement calculated with the Sekiguchi-Ohta model was 31 mm in 10,000 years and 33 mm in 100,000 years. The long-term effectiveness of the colloid filtration and self-sealing properties of the buffer was then evaluated as follows.

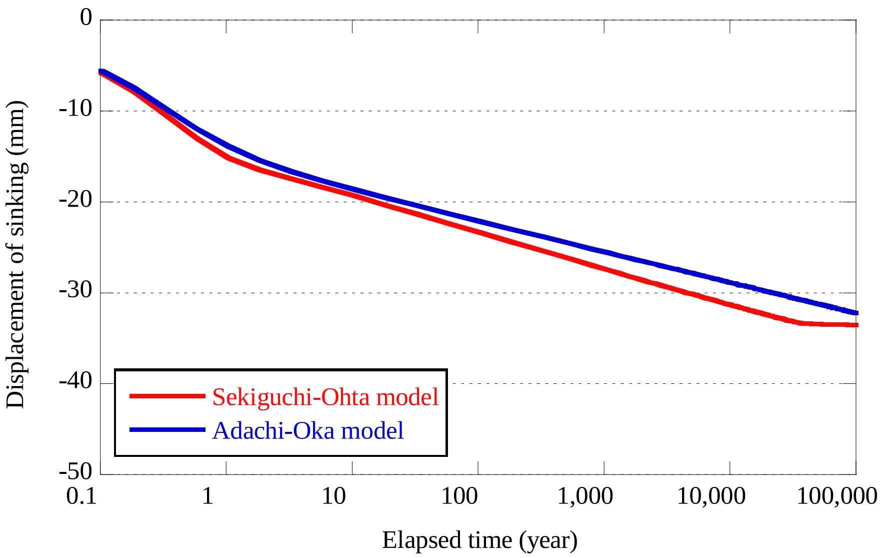


Figure 2.2.4-14 Results of overpack sinking analysis

The sinking of the overpack and/or deformation of the buffer may cause air gaps in the upper part of the engineered barrier system. In this case, it would be important for the buffer emplaced above the overpack to have a sufficient density to swell and fill the generated air gaps. For this evaluation, the volumetric swelling ratio $\overline{V}$ was used to determine the increase in the volume of the buffer after swelling compared to the original volume, and to check whether the obtained value is sufficient for the buffer to fill the generated air gaps.

Assuming V_1 as the air gap volume, V_2 as the volume of the buffer emplaced above the overpack and ρ_e as the effective clay density, after 100,000 years V_1 and V_2 were calculated as the sum of the deformation for each mesh element and ρ_e was set conservatively to the minimum value of the mesh elements for the buffer on the overpack. Table 2.2.4-6 shows the volume of the engineered barrier system components and the effective clay density after 100,000 years.

Table 2.2.4-6 Volume of engineered barrier system component and effective clay density 100,000 years later

Constitutive model	Volume of pore [m^3] V_1	Volume of buffer [m^3] V_2	Volume ratio [−] $(V_1+V_2)/V_2$	Effective clay density [$Mg\ m^{-3}$] ρ_e
Sekiguchi-Ohta model	0.220	6.489	1.034	1.320
Adachi-Oka model	0.247	6.458	1.038	1.322

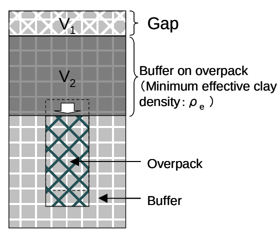


Figure 2.2.4-15 shows the result of the deadweight sinking analysis, plotted on the graph indicating the relationship between effective clay density and volumetric swelling ratio $\overline{V}$ obtained from the gap swelling tests for the buffer (Kikuchi and Tanai, 2005). The volumetric swelling ratio was calculated here by dividing the maximum buffer volume after swelling by the original buffer volume. The result of the gap swelling test shown in the figure is not for Horonobe groundwater but for simulated seawater with a higher ionic strength than the Horonobe groundwater. A volumetric swelling ratio of 1.038 is required to fill the air gap calculated in the deadweight sinking analysis (Adachi-Oka model). Figure 2.2.4-15 shows that a ratio of 1.092 can be achieved even for the simulated seawater (test period of 2 weeks). The result for a test period of 6 months with the simulated seawater shows an even higher ratio. From this, it can be concluded that buffer with the specifications used for this analysis has sufficient self-sealing to fill the air gap generated by the deadweight sinking of the overpack.

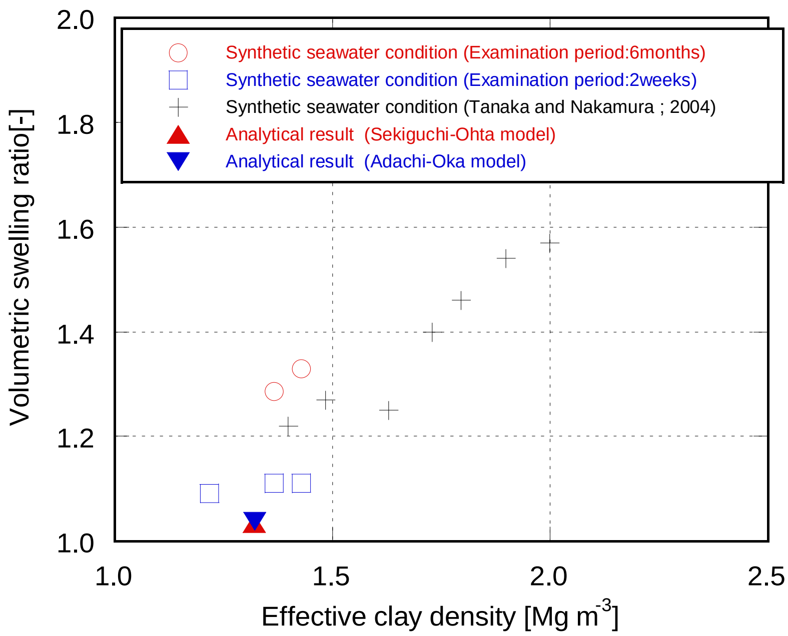


Figure 2.2.4-15 The volumetric swelling ratio of buffer and analytical result of overpack sinking

In the case where the buffer above the overpack swells to fill the air gap generated by overpack sinking, the density of the buffer will decrease. Even in this situation, the density of the buffer is still required to remain above 0.80 Mg m^{-3}, which is the lower limit of effective clay density that ensures the colloid filtration function. The effective clay density after 100,000years of swelling of the buffer is calculated by dividing the effective clay density by the volume ratio. Calculated using the values shown in Table 2.2.4-6, the buffer density after swelling should be 1.27 Mg m^{-3} for both models. Thus, it can be concluded that the buffer should maintain its colloid filtration function even after 100,000 years.

The degree of critical erosion of the buffer immediately after corrosion expansion of the overpack (after 4,750 years) obtained by the corrosion expansion analysis is shown in Figure 2.2.4-16. This indicates that the buffer as a whole does not reach a critical state and should remain mechanically stable, even though the stress almost reaches a critical state around the overpack and at the corners.

To summarize the result of the evaluation of the long-term mechanical deformation of the buffer by analyzing overpack deadweight sinking and corrosion expansion, no phenomena have been observed that would compromise the long-term integrity of the buffer.

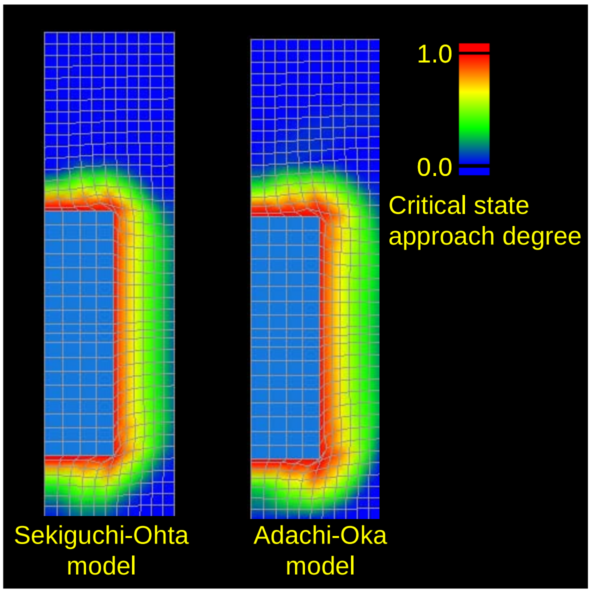
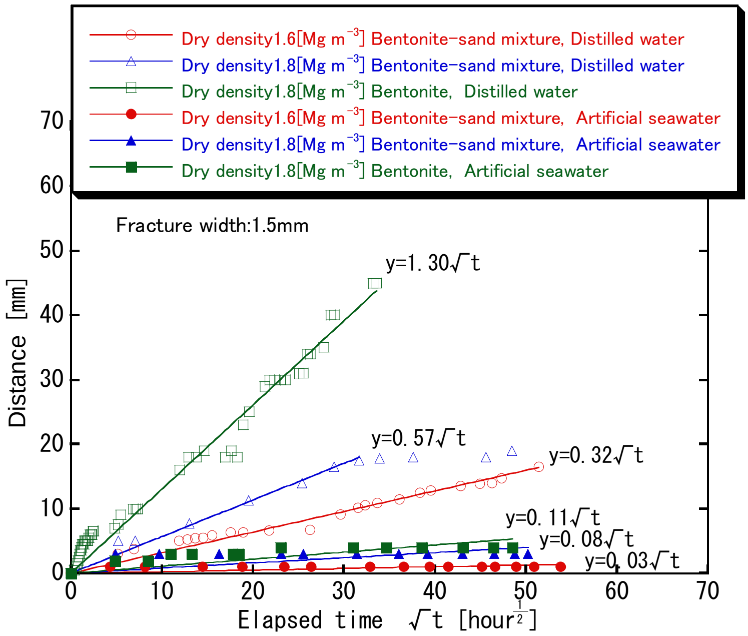


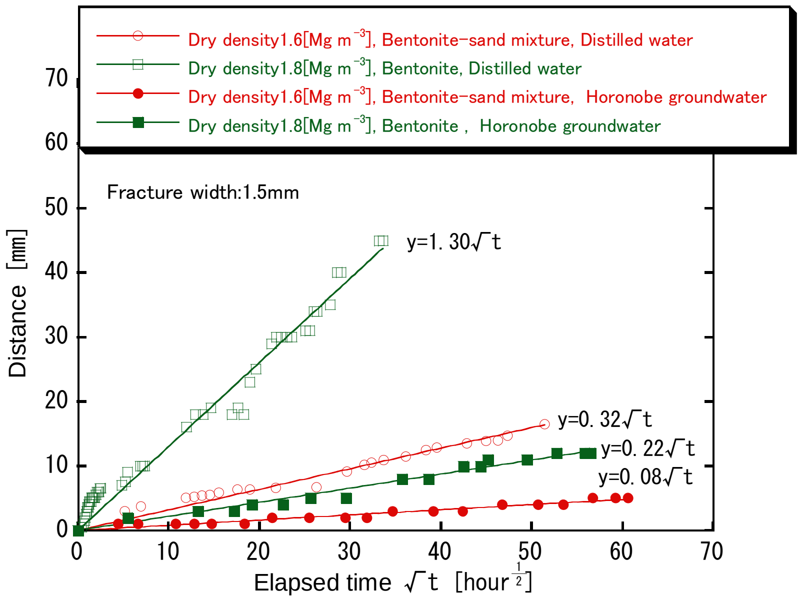
Figure 2.2.4-16 Distribution of degree of critical state approach (4,750 years later)

3) Evaluation of extrusion of the buffer material

Extrusion of the buffer material into fractures in the rock has been evaluated in tests using buffer material with the specifications as outlined above (section 2.2.4 (2)) and simulated fractures, assuming saline groundwater conditions (Horonobe groundwater and simulated seawater). The result (Figure 2.2.4.-17) shows a smaller extrusion rate for saline groundwater than for freshwater (distilled water) by a factor of four (for the Horonobe groundwater) to ten (for simulated seawater) (Matsumoto and Tanai, 2004 ; 2005). It is reported that the swelling pressure is lower in the saline groundwater due to agglomeration of the bentonite (Matsumoto and Tanai, 2005) and, since swelling is the driving force for the extrusion phenomenon, the reason for the difference is assumed to be the lower swelling under saline groundwater conditions. It has been confirmed that the reduction in the density of the buffer is smaller in saline groundwater than in freshwater. Therefore, it is unlikely that this phenomenon will become a significant problem under the Horonobe groundwater conditions.



(a) Comparison of distilled water and synthetic sea water



(b) Comparison of distilled water and Horonobe groundwater

Figure 2.2.4-17 Change of buffer outflow distance with time (Matsumoto and Tanai, 2005)

4) Evaluation of gas migration

For the evaluation of the effects of hydrogen gas generated by corrosion of the carbon steel overpack, the increase in pore pressure and gas release rate were analyzed using the improved TOUGH2 code, based on the porosity, density and thermal properties of rock at the depth of interest described in section 2.2.1(1) (Matsui et al., 2005a). Since no measured data were available for gas migration parameters such as absolute or relative permeability, these parameters were determined based on the results of gas migration tests for tuff (Matsui et al., 2005a). The corrosion rate was set at 10 μm y^{-1}, the same value as in the H12 Report (JNC, 1999). The result indicates that the pore pressure will increase to 5.7 MPa after three years, which will cause gas migration into the buffer. Figure 2.2.4-18 shows the distribution of gas saturation when gas migration starts. Migration of gas into the rock will start after approximately 100 years. After approximately 130 years, the pore pressure at the interface of overpack and buffer will reach almost a steady state at 4.9 MPa. Figure 2.2.4-19 shows the distribution of gas saturation after 128 years. It indicates that the gas flows through the side wall of the disposal pit and then migrates into the rock around the disposal tunnel. The gas release rate into rock reaches 0.08 kg y^{-1} after a short time and then decreases to approximately 0.02 kg y^{-1}, at which rate the release would continue.

The findings of the analyses can be summarized as follows:

- The maximum increase in pore pressure will be approximately 1.5 MPa.

- The cumulative amount of porewater displaced from the buffer into the rock associated with gas migration will be a maximum of approximately 8%. The change in the degree of saturation of the buffer should therefore be very small.

- The gas release rate into the rock will reach a peak immediately after it starts, but continues for only a short period of time. After this, release continues at a constant rate that is almost the same as the gas generation rate. Therefore, accumulation of gas between the buffer and the rock is unlikely.

It is therefore concluded that there will be almost no mechanical effects of gas generation on the engineered barriers under the conditions assumed in this study.

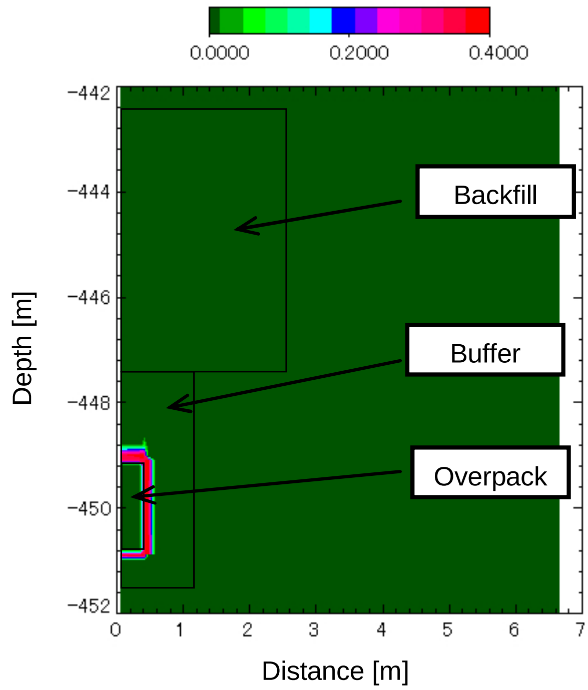


Figure 2.2.4-18 Distribution of degree of gas saturation after 3 years

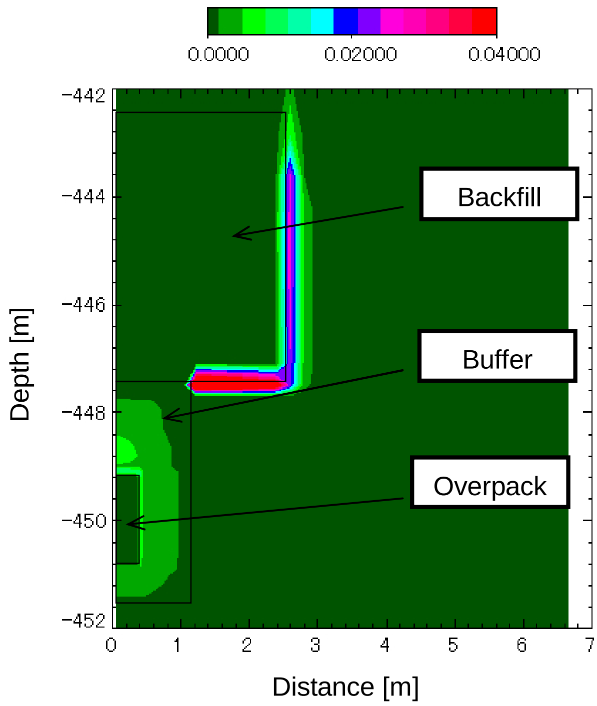
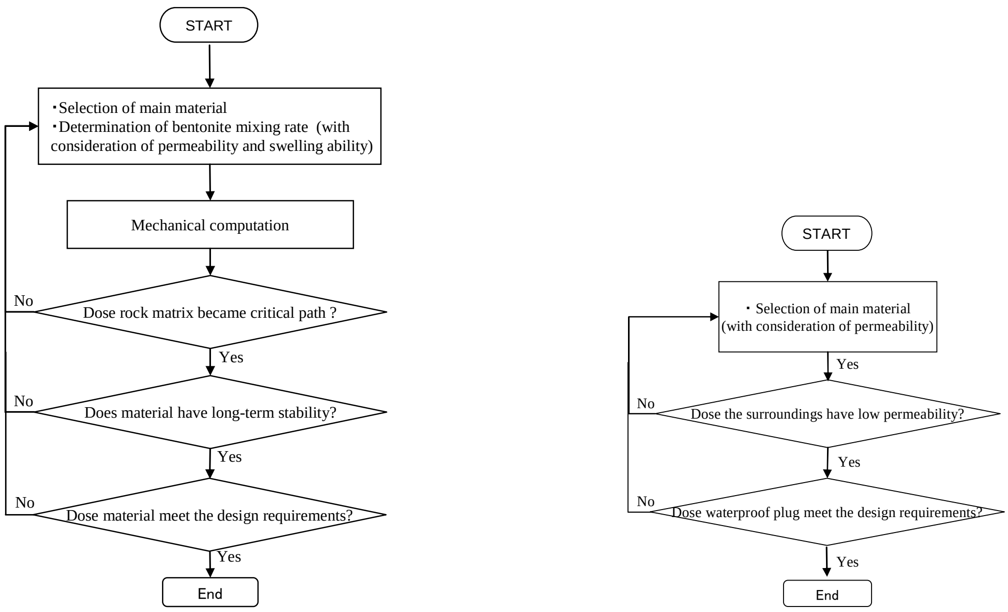


Figure 2.2.4-19 Distribution of degree of gas saturation after 128 years

2.2.5 Application of the backfill design method

Tunnels are a temporary means of transport for the installation of the engineered barrier system. After the system has been installed, these tunnels must be filled and sealed without damaging the integrity of the system. If tunnels excavated to construct a repository are left open and unattended, they may have significant adverse effects on the barrier performance of an entire repository. For example: the mechanical stability of a tunnel may be damaged by rock stresses and an open tunnel may provide a fast pathway for groundwater flow. If excavations connecting underground facilities directly with ground installations are left open, they may provide a channel linking a repository with the human environment and people may inadvertently intrude into the repository. Appropriate measures therefore have to be taken, such as backfilling and/or plugging, in order to avoid significant impacts on the overall performance of the geological disposal system (JNC, 1999). Two points should be taken into account in designing the closure system: the backfill itself does not constitute the EBS and there will be a certain time lag between the installation of the EBS and backfilling. The individual specifications should be defined according to the requirements of the total system, combing backfill and plugs, in order to ensure the safety of the repository based on the given geological environment. Figure 2.2.5-1 shows an example of the design flow for the backfill and sealing plugs using information on the geological environment at Horonobe; details are described below.



(a) Backfill material (b) Waterproof plug

Figure 2.2.5-1 Design flow of backfill and waterproof plug considerd actual geological conditions

(1) Design of the backfill

The total tunnel length in the entire underground facilities of the repository would amount to more than 100 km, producing a large amount of rock from excavation. The H12 Report concluded that this rock should be used as the main material for the backfill because it originates from the site and is readily available and economical. The specifications were defined as a bentonite mixing ratio of 15 wt% and a dry density of 1.8 Mg m^{-3} (effective clay density of 0.6 Mg m^{-3}) (JNC, 1999). The intrinsic permeability of backfill with these specifications would be 10^{-18} m^2 and swelling pressure approximately 0.10 MPa under freshwater conditions. Adjusting the grain size of the rock and mixing it with bentonite could result in a low permeability and, in addition, the self-sealing function would also be expected to fill the gap between the backfill and the tunnel wall because of the swelling of the bentonite. Bentonite-mixed excavated rock was used for the backfill under the geological conditions at Horonobe in this study. The results are summarized below.

In order to achieve the same level of intrinsic permeability as for the buffer with the specifications described above (10^{-18} m^2), an effective clay density of approximately 1.2 Mg m^{-3} would be required, based on the results of permeability tests carried out for the Horonobe groundwater (Figure 2.2.5-2). The test results for swelling pressure shown in Figure 2.2.5-3 also suggest that an effective clay density of approximately 1.2 Mg m^{-3} would be required. It should be noted that, in Finland, the swelling of the backfill is expected to perform a gap-filling function, as well preventing blocks in the tunnels from falling down; the swelling pressure required to maintain stability would be several kPa (Autio et al., 2002). Backfill with an effective clay density of approximately 1.2 Mg m^{-3} should have a swelling pressure of several hundred kPa, which will allow the backfill to perform the functions expected in the Finnish study. In the case of vertical emplacement, when the dry density of the backfill is low, swelling extrusion of the buffer material may occur, but backfill with an effective clay density of 1.2 Mg m^{-3} or more should prevent swelling extrusion of the buffer material, because it is expected that equilibrium swelling pressure of the buffer should be smaller with Horonobe groundwater than freshwater. The results from a three-dimensional elastic analysis by Sugita et al. (1999) under freshwater conditions also indicate that extrusion will not occur with this effective clay density. With regard to the requirement that the backfill should not extrude over a long period of time, it has been demonstrated that extrusion is not as significant for saline groundwater as for freshwater (Matsumoto and Tanai, 2005).

Based on the above discussions, the specifications of the backfill for the groundwater conditions at Horonobe were determined to be an effective clay density of 1.2 Mg m^{-3}, a bentonite mixing ratio of 40 wt% and a dry density of 1.8 Mg m^{-3}.

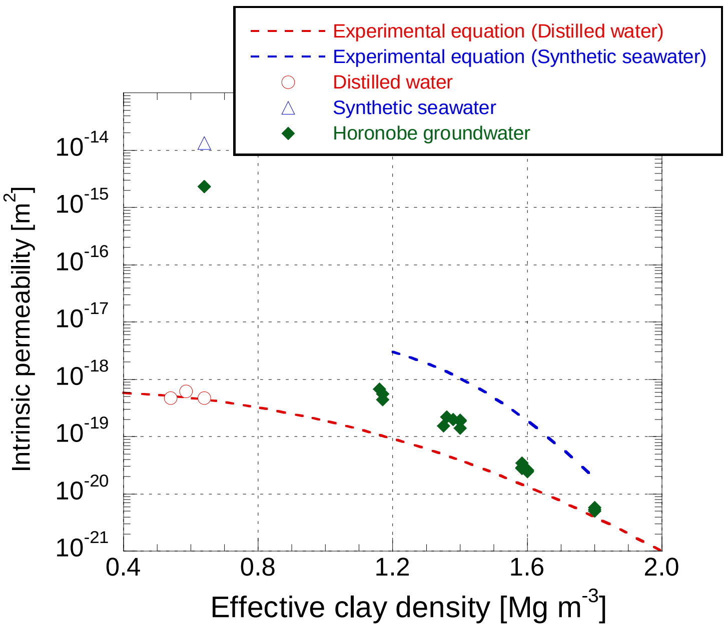


Figure 2.2.5-2 Relationship between effective clay density and intrinsic permeability (Kikuchi and Tanai, 2005)

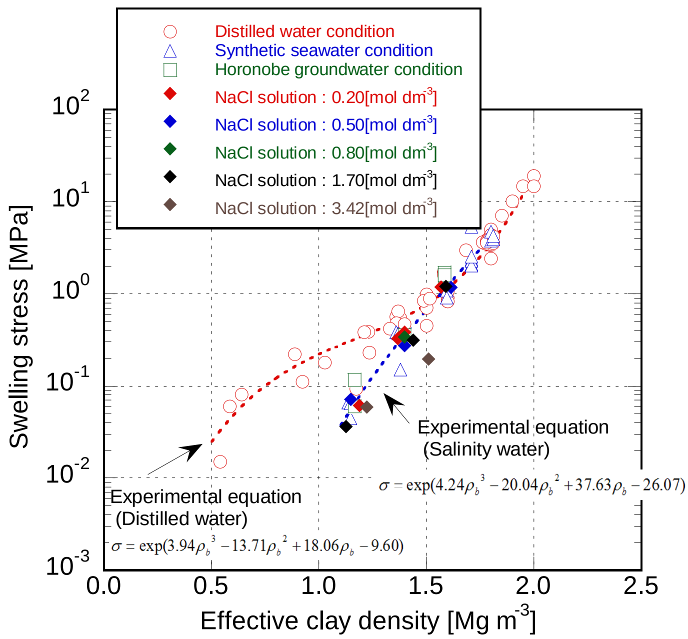


Figure 2.2.5-3 Relationship between effective clay density and swelling pressure (Kikuchi and Tanai, 2005)

(2) Design of the plugs

The H12 Report concluded that material with a high bentonite content (similar content to that of the buffer) would be suitable for the plugs, with the aim of achieving long-term sealing performance. An example of a plug concept for sedimentary rock used for the disposal of Category B waste (intermediate-level radioactive waste) in France is shown in Figure 2.2.5-4 (ANDRA, 2005). This concept uses swelling clay to realize low permeability and plasticity of the plug itself and a plug structure in the tunnel to isolate the EDZ.

This concept is used also in this study, i.e. a chemically stable material with a high bentonite content equivalent to that of the buffer is used so that the plug has a low permeability and a plug structure isolates the EDZ. The effective clay density of the plug used in a tunnel sealing performance test (Martino et al., 2003) conducted as a joint study with AECL (Canada) was equivalent to 1.6 Mg m^{-3}. The intrinsic permeability of the clay for the Horonobe groundwater would be within one order of magnitude higher than that for freshwater (Figure 2.2.5-4). The sealing plug should therefore be composed of clay blocks and a shot-clay layer (Martino et al., 2003), with a sand mixing ratio of 30 wt% and a dry density of 1.9 Mg m^{-3} for the clay blocks and 1.3 Mg m^{-3} for the shot-clay layer (Figure 2.2.5-5). As is often seen for sedimentary rock, it is expected that the rock zone that has loosened at the Horonobe URL site will recover due to creep deformation and reaction force from the inside of the tunnel (Okubo, 2005). In this case, the permeability of the EDZ would also recover, allowing more efficient plug design.

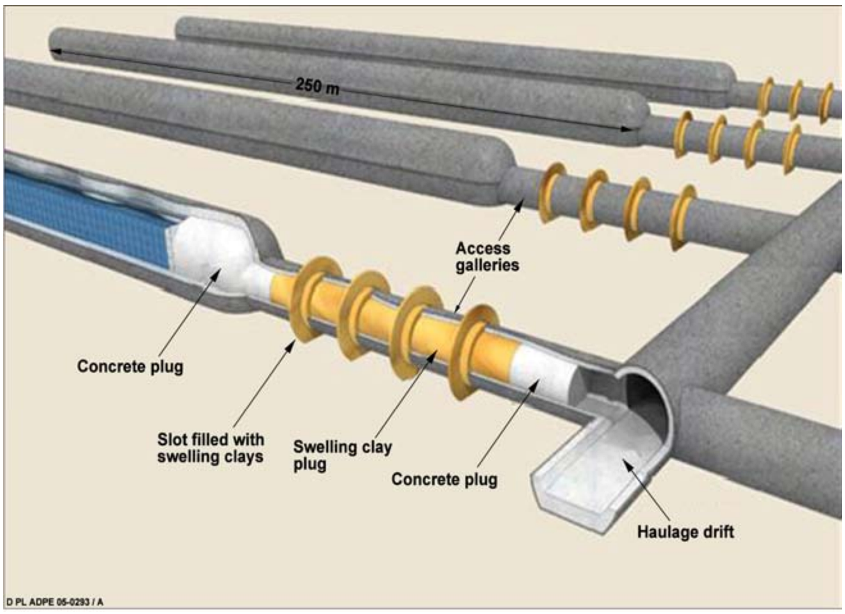


Figure 2.2.5-4 French concept of pluging in sedimentary rock

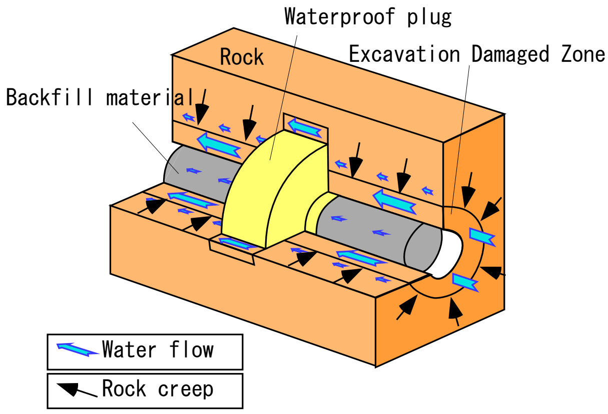


Figure 2.2.5-5 Concept of pluging in Horonobe

In this study, the example of applying the design method was based on the design requirements for the closure system described in section 2.1.3 and on the geological conditions at the Horonobe URL site. As a result, it could be demonstrated that the sealing plug used in crystalline rock could also be used in a saline groundwater environment.

The Swedish study SR97 indicated that the backfill would be one of the weak points in the disposal concept, particularly from the long-term perspective (Sellin, 2002). For example, the influence of ion exchange is not an insignificant problem for backfill with a low bentonite content. It can lead to reduced swelling of the backfill under the highly saline groundwater conditions. The result of an extrusion test for buffer material shows that this should not cause a serious problem (Matsumoto and Tanai, 2005), but if extrusion were to proceed further the swelling capability would be reduced and a gap would be generated, which could function as a groundwater flowpath. Although increasing the bentonite content of the backfill may counteract these potential problems, and should improve swelling and hydraulic properties, it would lead to increased material costs and less rigidity. An alternative measure in terms of cost and performance would be to use natural clay materials, which would, however, require sufficient investigation of their properties. Constructing a repository at an inland location would improve the likelihood of having groundwater with lower ion concentrations, thus avoiding the problems associated with saline groundwater (Sellin, 2002). For the engineered barrier concept, horizontal emplacement concept would also be an alternative (Sellin, 2002). It will be important to evaluate the functions of the individual components in the context of the overall repository layout to make the function required of each component more clear; this will lead to a more reliable and rational closure concept. A relevant study is being carried out in cooperation with NUMO and is evaluating various combinations of backfill and plugs (Sugita et al., 2005).

2.2.6 Development of low alkaline concrete

(1) Requirements on cementitious materials for the construction of disposal facilities

Constructing tunnels at a depth of several hundred meters often requires installation of rock supports to ensure mechanical stability of caves. Ordinary Portland cement (OPC) has been widely used for rock supports, which would however result in highly alkaline groundwater with a pH of 12.5 to 13. For the HLW repository, it is a matter of concern that the highly alkaline groundwater could cause alteration of the buffer and the surrounding rock, with a resulting adverse effect on the performance of the engineered and natural barriers. These effects have been evaluated internationally (NUMO, 2004b). If these studies indicate that highly alkaline groundwater would have a significant impact on the safety of the repository, development of alternative materials would be necessary to avoid such impact.

The influence of high-pH groundwater due to use of OPC is evaluated in terms of the extent and period of alteration of bentonite and surrounding rock, using the concentration and amount of highly alkaline leachate from the concrete as parameters. Low alkaline cement, low leachability concrete and low permeability bentonite have been studied as measures for limiting undesirable influences. Since the chemical environment in a geological disposal facility would be determined by very long term reactions, this study focuses on low-pH cement, which would ensure a low alkaline condition even over the long-term.

Of the materials that have been widely used in infrastructures, concrete mixed with pozzolanic materials such as silica fume or fly ash has the advantages of increased long-term strength, reduced heat of hydration, reduced bleeding water and increased durability. On the other hand, its early strength and pH value tend to be low. JAEA therefore developed a new type of concrete by adding large amounts of pozzolanic materials to OPC in order to limit the high alkalization of groundwater; this new cement is called high fly ash silica fume cement (HFSC). Compared to concrete using OPC alone, HFSC concrete is expected to have a low alkaline leachate. This type of cement is referred to here as low alkaline cement and the concrete produced using it as low alkaline concrete.

The H12 Report proposed the use of HFSC for rock supports and specified the development targets as follows.

(a) The pH of the leachate from the cementitious materials should be 11 or lower.

Tests were conducted to determine the alteration of the granite using a simulated cement leachate with a pH of 11, 12.5 and 13. The result indicates that leachate with a pH of 12.5 or higher could cause reactions such as dissolution of the granite, while that with a pH of 11.0 or lower causes smaller effects in terms of degree and range (Owada et al., 2000). Other tests conducted for bentonite with a simulated cement leachate with pH 10.5, 11.5, 12.5 and 13 indicate that dissolution of the bentonite would be greater at a pH of 11.5 or higher, while significant changes could be avoided at a pH of 10.5 or lower (Kubo et al., 1998). The H12 Report therefore concluded that a pH of 11 would be the maximum value for cement leachate at which significant alterations could be avoided, although data on long-term behavior should be collected to allow proper evaluation of influences on the surrounding rock, bentonite-based materials and the corrosion mode of the overpack.

The lower limit of pH that can be realized for cementitious materials is assumed to be between 10 and 11, from the viewpoint of technical feasibility. These considerations led to a target pH value of 11 for HFSC.

(b) Various construction methods should be used flexibly, such as shotcrete, in-situ casting and precast concretes for rock supports.

The potential use of cementitious materials in and around a HLW repository includes those for shotcrete, in-situ cast and pre-cast concretes for the rock support, concrete cast in-situ for the mechanical plug, grouting as filling material for fractures and rock bolt holes as shown in Figure 2.2.6-1.

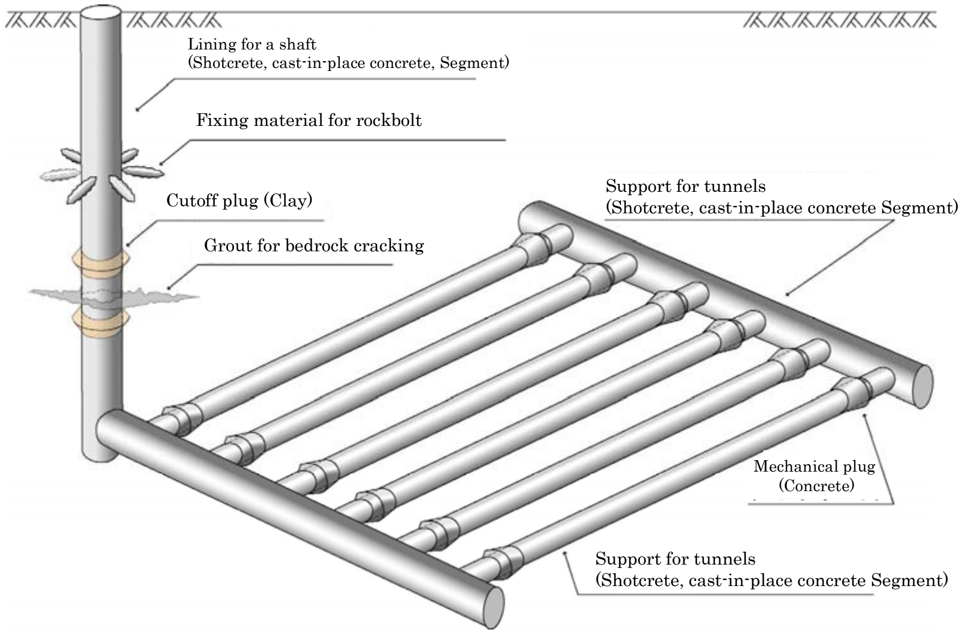


Figure 2.2.6-1 Concept of an artifact with possibility
to use cementitious materials for HLW repository

Described below are the requirements for cementitious materials made since publication of the H12 Report. The requirements of the implementing organizations (NUMO, 2004b) include:

(a) Cementitious materials should have sufficient construction performances and mechanical properties.

Concrete to be used for support has to be strong enough to maintain tunnel stability. It should also have sufficient workability to allow shotcreting or casting.

(b) Cementitious materials should have the required low alkalinity.

Table 2.2.6-1 shows different types of low alkaline cements under development in various counties. Some other types are being developed, using pozzolanic materials for example (SKB, 2003).

Table 2.2.6-1 Various types of low-alkaline cement

	HFSC JNC(Japan)	LHHPC(Gray and Shenton, 1998) AECL (Canada)	LAC-S (Fujita et al., 1998) CRIEPI(Japan)
Mechanism of low pH	Pozzolanic reaction	Pozzolanic reaction	cement clinker formulation
cement	OPC:40%	OPC:25%	LAC-C:80%
Silica fume	20%	25%	20%
Fly ash	40%	0%	0%
Silica flower	0%	50%	0%
pH	Less than 11.0	10.6	10.2
Compressive strength at 28days	More than 40MPa (Target)	More than 70MPa	More than 40MPa

OPC：Ordinary Portland Cement

LAC-C：Hauyne（$3CaO \cdot 3Al_2O_3 \cdot CaSO_4$）based cement clinker

HFSC was developed with the aim of lowering the pH of the leachate from hardened cement by reducing the OPC content, i.e. by increasing the content of pozzolanic materials (fly ash and silica fume), so that the calcium hydroxide that causes high pH will be consumed by pozzolanic reaction to produce calcium silicate hydrate (CSH) with low solubility and a low Ca/Si ratio. If 70% or more of OPC is replaced with silica fume, which has very fine particles, the material would have a high viscosity and reduced workability when mixed with water. Therefore, 30% or more of the silica fume was replaced with fly ash to limit the mixing ratio of the silica fume to within 20% (Iriya and Mihara, 2003). However, the fly ash is difficult to find with consistent quality, which would result in variable quality of the concrete properties. In addition, a reduced OPC content leads to low early strength, compared with concrete using OPC alone. The reduced pH can also cause corrosion of rebars in the reinforced concrete.

As described above, there are many issues to be addressed in the future regarding HFSC, associated mainly with its high content of pozzolanic materials. There is also a need for low alkaline concrete in a HLW repository. In this study, we conducted material tests for HFSC with different mixing ratios of pozzolanic materials in order to understand its properties and to determine the specifications required for achieving the above targets.

Table 2.2.6-2 shows the basic HFSC mixing ratios tested in this study.

Table 2.2.6-2 Basic mixture of HFSC

	Ordinary Portland Cement	Pozzlanic material	
		Silica fume	Fly ash
OPC	100%	0%	OPC
HFSC424	40%	20%	HFSC424
HFSC325	30%	20%	HFSC325
HFSC226	20%	20%	HFSC226

(2) Evaluation of construction performance and compressive strength

Concrete using HFSC has a different workability to that using OPC and there is as yet no actual engineered structure where HFSC concrete has been used. Good construction performance in the actual disposal environment would be essential for the concrete to be used in the construction of disposal repositories because of the long transport distance to the tunnels at a depth of several hundred meters and the long time required for the transport. Since the concrete is also used as structural material that requires a certain strength, such as shotcrete for the rock supports, mechanical properties such as compressive strength that meet the design requirements are also necessary.

Construction performance and compressive strength were evaluated for precast concretes and shotcrete. Concrete will be used as support during excavation with a tunnel boring machine (TBM) in the horizontal emplacement concept (JNC, 1999). Shotcrete will be used as rock support for the New Austrian Tunneling Method (NATM) in the vertical emplacement concept (JNC, 1999).

1）Manufacturing and compressive strength test of concrete segments

(a) Description of the test

Fine-particle silica fume and fly ash contained in HFSC increase its viscosity when mixed with water, which may reduce workability for pumpability or compacting. In order to improve the workability of HFSC concrete, a superplasticizer needs to be added. Sample segments were manufactured using HFSC concrete with high fluidity to evaluate the workability and were then subjected to loading tests.

(b) Test conditions

Since the number of the sample segments that could be manufactured was limited, the binder type HFSC226 was selected for the tests; this has the largest content of pozzolanic materials and thus the lowest pH. The water-binder ratio was set at 27.3% to see how good the increased fluidity would be for compacting into the segment form.

The water-binder ratio means the ratio between the weight of water and binder (including cement silica fume and fly ash) as a percentage. Generally speaking, the smaller the ratio, the lower the fluidity due to less water, but the higher the strength due to higher density. The superplasticizer used in this study was the commonly used polycarboxylic acid-based type (Iriya and Mihara, 2003). The admixture, a surface acting agent, may form a complex with radionuclides and it is also confirmed by experiments that it will increase radionuclide solubility (Greenfield et al., 1998). However, the admixture would not have a large impact on radionuclide migration because it would be strongly adsorbed onto the solid phase of the cement and the molecular weight of the admixture leached from the cement paste after hardening would be very different from that before the admixture is mixed into the concrete (The Federation of Electric Power Companies of Japan and JAEA, 2007).

Circular arc test specimens with an outer diameter of 5,600 mm, a thickness of 250 mm and a width of 1,200 mm were produced by casting HFSC226 concrete into a steel form without compaction; this is then steam curing at a high temperature (60° C for three hours) and removed of the form. The segments were cured in air or in water for one year. After this, they were subjected to bending fracture tests by two-point concentrated loading.

(c) Test results

A test using the full-scale segments confirmed that concrete using HFSC226 with high fluidity had good compacting properties, one of indicators for workability. The segments were cured for one year in air or water and subjected to bending fracture tests. Both segments showed sufficient strength required from design against initial cracking and bending fracture. These properties were also comparable to those of ordinary concrete (Nakayama et al., 2004).

2) Shotcreting test using a mock-up tunnel

(a) Description of the test

HFSC needs to be used with an accelerator, since, similarly to OPC, it tends to be difficult to realize the early strength required for the shotcrete. A shotcreting test was performed in a mock-up tunnel constructed ground surface using HFSC concrete to which an accelerator was added, in order to evaluate its workability and strength.

Another concern was that, although a superplasticizer was added to HFSC to improve workability, the fluidity (slump) could decrease with time due to adsorption of the unburned coal contained in the fly ash. Reduced fluidity can have an adverse impact on workability, e.g.

pumpability. Therefore, change of fluidity with time was also measured for the fresh HFSC concrete.

(b) Test conditions

Specimens were prepared using OPC and early-strength cement with a binder mixing ratio which is basically the same as that for HFSC424; these were named HFSC424N and HFSC424H respectively and their early strengths were compared. The water-binder ratio was set as low as possible so that a certain early strength could be assured, i.e. 40% for HFSC424N and 45% for HFSC424H (Konishi et al., 2006). For the accelerator, a calcium sulfoaluminate-based inorganic material with pH value between 12 and 13 was used for OPC and with a pH value between 10 and 11 for HFSC (Iriya et al., 2001).

Shotcreting performance was evaluated by visual inspection and by touching to check the degree of mixing of the accelerator, pulsing motion of the pumping tube and amount of dust.

For the compressive strength tests, the early strength of the concrete specimens at ages of 3, 6 and 24 hours was obtained by converting the compressive strength obtained by a pullout test. For the concrete at ages of 7, 28 and 56 days, concrete with the accelerator was sprayed onto the panel form and concrete cores were formed with 50 mm diameter and 100 mm height and subjected to standard compressive strength tests.

The change of fluidity with time was measured for HFSC424N samples with water-binder ratios of 40% and 45% immediately after mixing and then 30 and 60 minutes later, respectively.

(c) Test results

As seen in Figure 2.2.6-2, no major difference was observed in shotcreting performance between HFSC424N and HFSC424H. Concretes with both compositions could be used for shotcreting.

Figure 2.2.6-2 HFSC shotcreting test using mock up tunnel

The results of the compressive strength tests are shown in Figure 2.2.6-3. Both compositions achieved the design strength required in the Horonobe URL (36 MPa at an age of 28 days). They also achieved a strength of over 3 MPa after 3 hours.

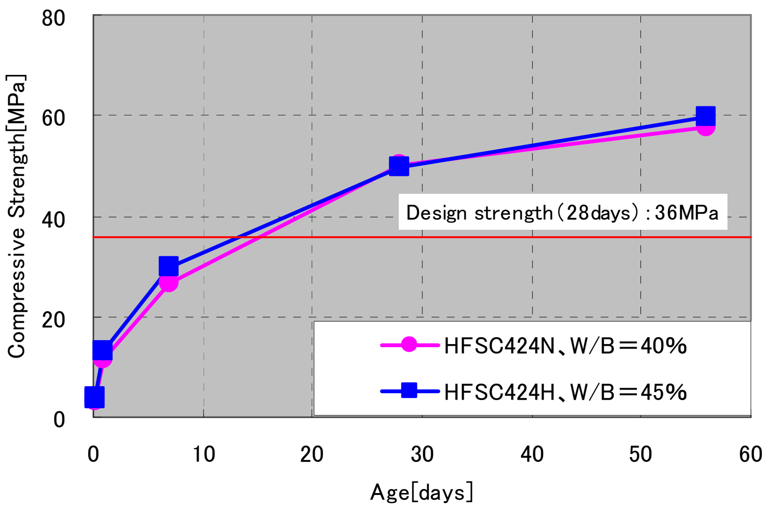


Figure 2.2.6-3 Result of compressive strength tests of shotcrete

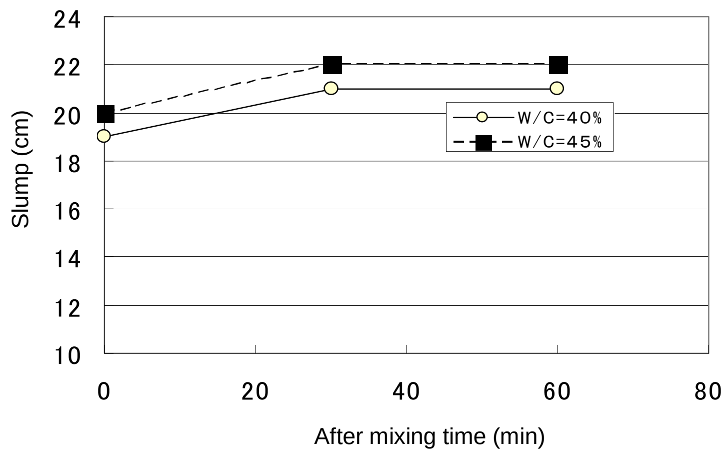


Figure 2.2.6-4 Slump change after mixing

The measurement results for change in fluidity with time are shown in Figure 2.2.6-4. The figure shows no decrease with time in slump after mixing, indicating that the reduction of fluidity of this concrete due to increased transport time would be small.

(3) Variability in the quality of fly ash

Fly ash to be mixed into HFSC is an industrial by-product of coal-fired thermal power plants and its quality varies depending on the type of coal used as the raw material. In this study, assuming application in the Horonobe URL, the variability in quality of the concrete was investigated using fly ash sampled from the same coal-fired thermal power plant over a period of two years. The slump flow before hardening and the compressive strength after hardening were used as measures of the quality of the concrete.

1) Slump flow test

(a) Description of the tests

The quality of HFSC concrete before hardening was evaluated using slump flow. Slump flow is an indicator of the consistency of the concrete. Concrete is put into a bottomless container placed on a plate, the container is then removed upwards and the fresh concrete spreads into a circle. The diameter of this circle is expressed in cm.

(b) Test conditions

The fly ash materials used in the tests were equivalent to JIS 1st grade and 2nd grade. The binder mixing ratio of HFSC424 was used to gain a relatively high strength. Assuming application as cast-in-place concrete that could be used for lining, etc., the water-binder ratio was set to 27.3% and superplasticizer was added to enhance the fluidity.

(c) Test results

The slump flow for HFSC424 cast-in-place concrete is shown in Figure 2.2.6-5.

The result of the first series of tests shows variability in slump flow in the range ±10 cm. The second series of tests conducted only for the JIS 2nd grade equivalent fly ash resulted in a range of 65 ± 5 cm, which is the recommended range for a high-fluidity concrete. This means that the variability in quality in terms of slump flow should be manageable for concrete with JIS 2nd grade equivalent fly ash by adding superplasticizer.

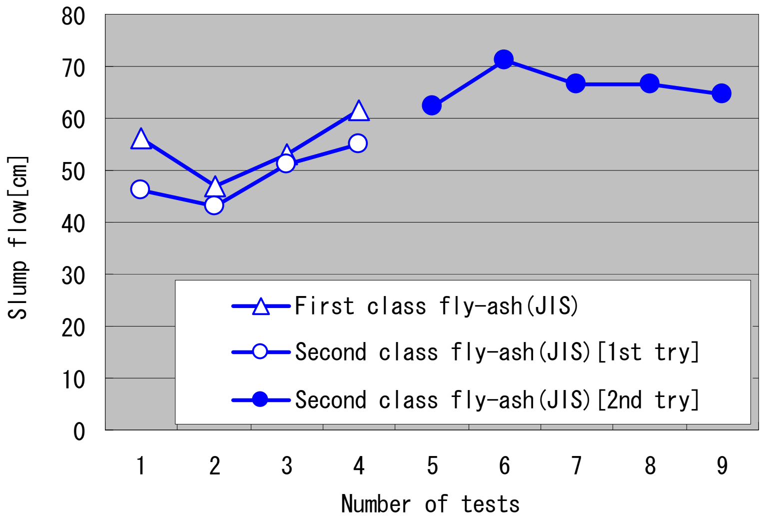


Figure 2.2.6-5 Variation of slump flow of cast-in-place HFSC424 concrete

2) Compressive strength tests

(a) Description of the tests

Fly ash and silica fume are being considered as binders in HFSC in order to restrict the increase in pH due to OPC. Therefore, given that the binder weight per 1 m^3 of concrete should remain the same, HFSC concrete would have a low compressive strength at an age of 28 days, compared to concrete using only OPC. In this study, the influence on compressive strength of variability in the quality of the fly ash and in the OPC mixing ratio in HFSC were evaluated. The target compressive strength is 40 MPa at an age of 28 days, which is the design strength specified in the basic plan for the Horonobe URL (Kubota et al., 2003).

(b) Test conditions

Two binder mixing ratios, HFSC424 and HFSC325, were used in the tests with a water-binder ratio of 27.3%. The fly ash was JIS 2nd grade equivalent and HFSC424 had the same composition as that used in the slump flow test shown in Figure 2.2.6-5 (the first series for JIS 2nd grade equivalent fly ash).

(c) Test results

The relationship between compressive strength and age is shown in Figure 2.2.6-6. Regarding the influence of the OPC mixing ratio, the compressive strength was higher for HFSC424, which contains more OPC, at each material age. Compared with the target compressive strength, HFSC325 also achieved the target value of 40 MPa and reached up to 47.2 MPa at an age of 28 days (average). The data fell in a range between 42.5 and 51 MPa, indicating that the quality of the fly ash tested in this study would not have a major impact on the compressive strength.

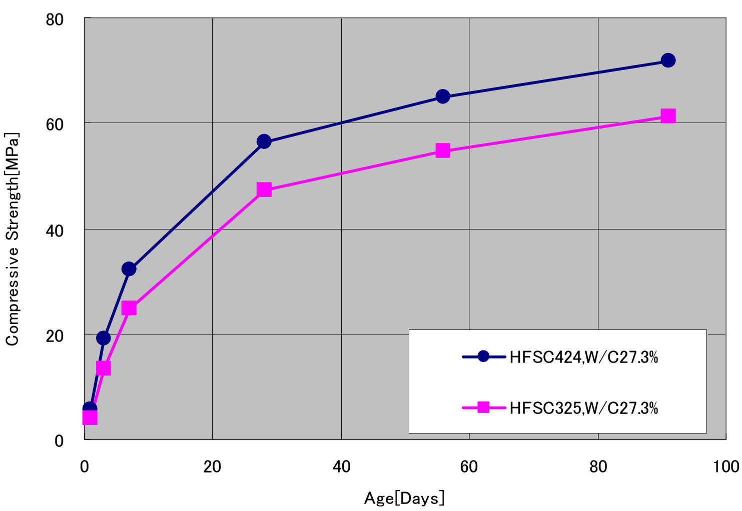


Figure 2.2.6-6 Relationship between compressive strength and age in HFSC concrete

(4) Evaluation of durability

1) Exposure tests under marine conditions

(a) Description of the tests

Not only cementitious materials such as shotcrete or lining (cast-in-place) concretes, but also steel for supports or rock bolts, will be used in a HLW repository. The concrete supports tend to function as compressive members and it is therefore unlikely that reinforced concrete structures will be used. Nevertheless, since one advantage of OPC is that its highly alkaline nature restricts the corrosion of rebars in the concrete, the influence of low alkalinity HFSC was evaluated by conducting exposure tests under marine conditions to determine the corrosion behavior of the rebars in the HFSC concrete.

(b) Test conditions

In order to limit the number of test specimens, two binder mixing ratios were selected, i.e. OPC only and HFSC226, where the water-binder ratio was set to 30% assuming use as cast-in-place concrete. The test specimens were 100 mm in diameter and 200 mm in height. Two round bars with 13 mm diameter were embedded in each concrete specimen at a location 20 mm from the surface. The test specimens were generally cured in water for 91 days and then immersed in the sea or placed in the splash zone above the sea in Shimizu Harbor, as shown in Figure 2.2.6-7. The progress of corrosion was monitored 0.5, 1 and 3 years after exposure started.

(c) Test results

The distribution of the chloride ion in the concrete specimens after three years of exposure is shown in Figure 2.2.6-8 and the corrosion status of the embedded rebars in Figure 2.2.6-9.

The rebar in the OPC concrete that was exposed for comparison purposes did not corrode significantly. On the contrary, 6 % of the surface of the rebar was corroded in HFSC226. Unlike the OPC, the rebars embedded in HFSC were corroded even though the chloride ion had not penetrated as far as 20 mm cover depth. This suggests that HFSC provides a more corrosive environment for the rebars than OPC.

Figure 2.2.6-7 Exposure test under marine condition with HFSC reinforced concrete

It should be noted that the corroded area of the rebar in HFSC did not increase from the first to the third year in the immersion tests, indicating that the progress of corrosion would be slow in HFSC. It cannot be concluded from the currently available data alone that HFSC would have a worse durability than OPC. Further data should be collected before conclusions are drawn.

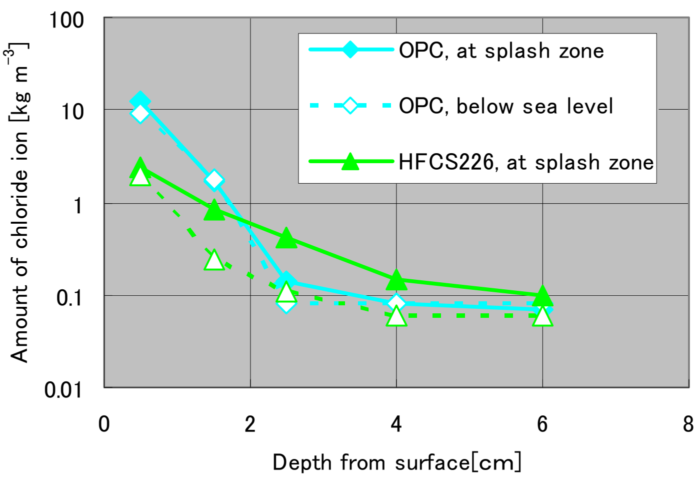


Figure 2.2.6-8 Chloride ion distributions in exposure test specimens 3 years later

Figure 2.2.6-9 Corrosion status of reinforcing bar embedded in HFSC226 concrete 3 years later

(5) Decrease in pH of HFSC

The decrease in the pH of HFSC was investigated by tests and analyses in order to determine the prospects of achieving the target pH of 11.

1) Immersion tests

(a) Description of the tests

Powdered HFSC paste was subjected to immersion tests to investigate the decrease in pH. The influence of temperature on the decrease in pH was also evaluated by increasing the temperature of the solution.

(b) Test conditions

The specimens were prepared by crushing hardened HFSC paste into powder. The specimens were immersed in distilled waster and the pH was measured. Since the pH can also be decreased by contact with carbon dioxide in the air, the immersion liquid container was filled with nitrogen gas. However, there will be contact with carbon dioxide gas during the measurement of pH and the influence of the carbon dioxide was also evaluated. The influence of carbon dioxide on hardened cement is generally known as carbonation, in which calcium carbonate is generated due to reaction of hydrates with carbon dioxide, and an X-ray diffraction analysis and differential thermal analysis were therefore conducted after the pH measurement. Almost no calcium carbonate was detected in these analyses, which would mean there was little influence of carbon dioxide in the air on the pH of the liquid (Iriya et al., 2003b).

Immersion tests were conducted at room temperature and at higher temperatures. The conditions for each test were as follows.

- Immersion tests at room temperature

The binder ratios for OPC as well as HFSC424, 325 and 226 were used. The water-binder ratio was set to 27.3%, assuming cast-in-place concrete. The test specimens, hardened cement paste powder with a particle size of 0.5 mm or less, were prepared by crushing cylindrical cement paste with dimensions 50 mm in diameter and 100 mm in height, and cured for 28 days in water. The powder was immersed in distilled water at a temperature of 20°C with a solid-water ratio of 0.5g ml^{-1} in order to achieve good agitation.

- Immersion tests at high temperature

The binder ratios of HFSC424 and 226 were used. The water-binder ratio was set to 40%, assuming shotcrete. The test specimens, hardened cement paste powder with particle size of 0.5 mm or less, were prepared by crushing cylindrical cement paste with dimensions of 50 mm in diameter and 100 mm in height, cured for 3 days cured in a sealed condition and dried in a vacuum. The powder was immersed in the distilled water at temperatures of 20°C and 65°C at a solid-water ratio of 0.5g ml^{-1} in order to achieve good agitation. The pH of the immersion liquid at 65°C was measured after it had cooled to 20°C.

(c) Test results

- Immersion tests at room temperature

The change in pH of the immersion liquid (distilled water) at a solid-water ratio of 0.5 g ml^{-1} is shown in Figure 2.2.6-10. For the powder specimens prepared from the paste with a water-cement ratio of 27.3 %, the immersion liquid for concrete containing only OPC showed a pH of approximately 12.8, which remained almost constant during the immersion period. On the other hand, the immersion liquid for HFSC226 that had the least OPC content showed a decrease in pH with immersion time, but the pH did not decrease to 11 even after 91 days immersion (Iriya et al., 2004b). For HFSC424 and 325 that had OPC contents between the above two cases, almost no difference was observed in the pH decrease rate between the two specimens; the decrease in pH started slightly earlier for HFSC325 than for HFSC424. In both cases, the decrease rate became slower when the pH had reached around 11.5 on the 182nd day and the pH did not decrease to 11 even after 476 days (Nakayama et al., 2004). Assuming that the decrease rate after the 182nd day, 7 x 10^{-4}/day, remains the same, it would take around 10 years for the HFSC424 immersion liquid to reach pH 11. Considering that actual concrete structures are not composed of powder, the pH decrease rate would be even slower.

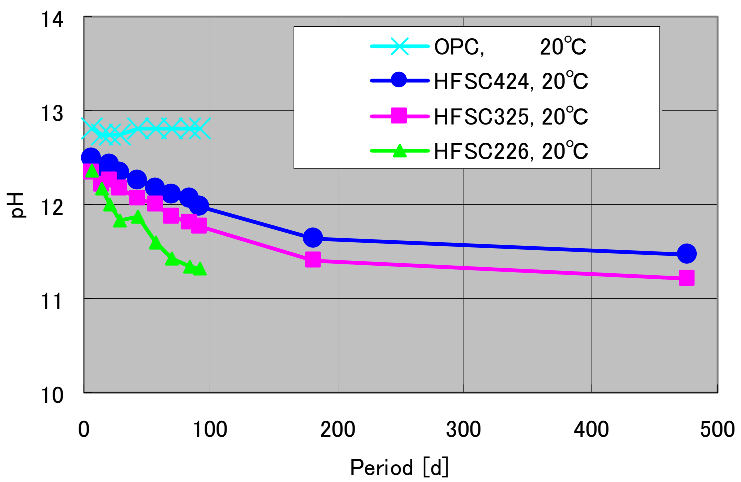


Figure 2.2.6-10 pH changes in soaking test under normal temperature with powdered specimens

- Immersion tests at high temperature

The change in pH of the immersion liquid (ion-exchange water) at a solid-water ratio of 0.5 g ml^{-1} is shown in Figure 2.2.6-11.

In the case where HFSC powder was immersed in the fluid at 65°C, the pH value decreased drastically up to the third day. In the case of HFSC226, it even reached a value lower than 11. The pH decrease rate then became smaller after the third day, as seen in the case of immersion at 20°C. This result suggests that the steam curing commonly used as a curing method for concrete products might be helpful in decreasing the pH to 11 or lower within a short period of time.

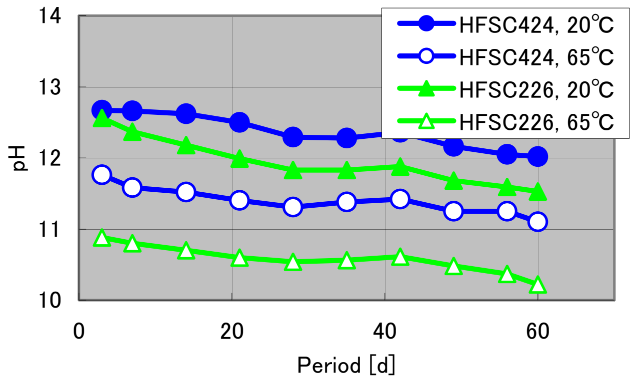


Figure 2.2.6-11 pH changes in soaking test under high temperature with powdered specimens

2) Analysis of the decrease in pH

(a) Description of the analysis

Since it proved difficult to confirm experimentally that the pH of the HFSC concrete would decrease to 11 or lower at room temperature, the evolution of the pH was evaluated analytically. The objective of this analysis was to reproduce the results of the powder immersion tests at room temperature shown in Figure 2.2.6-10.

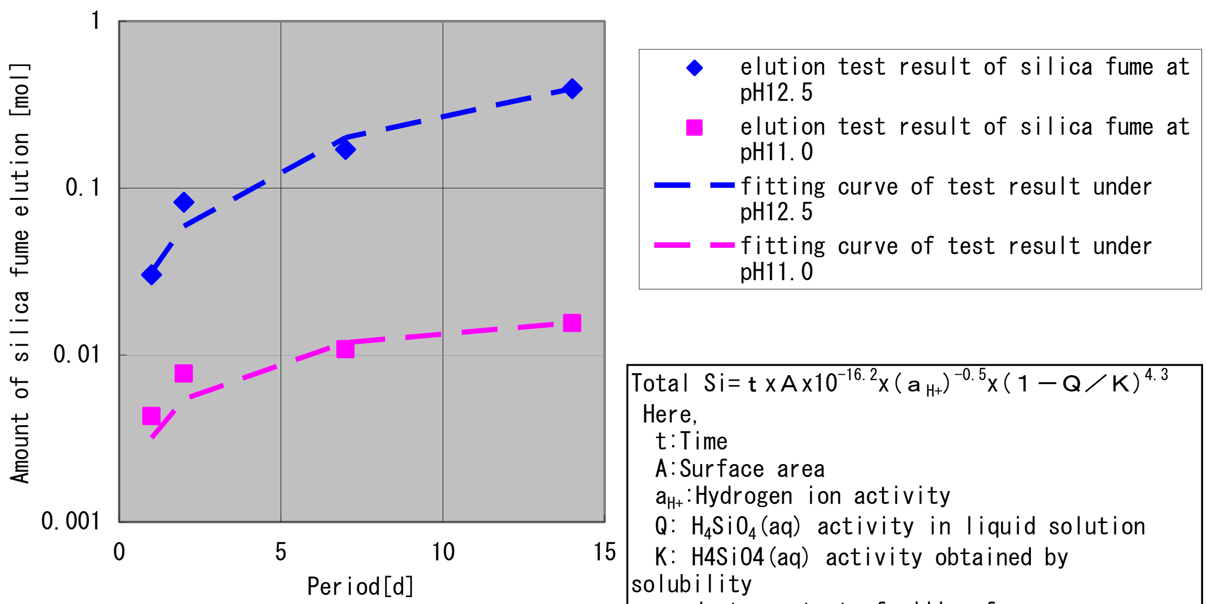
(b) Analysis conditions

The decrease in pH of HFSC depends on the pozzolanic reaction of silica fume and fly ash. In the pozzolanic reaction, highly alkaline portlandite ($Ca(OH)_2$) formed by the hydration of OPC reacts with the siliceous content (SiO_2) of the pozzolanic materials and forms CSH (calcium silicate hydrate) gel. Therefore, the analysis model included the reaction that causes pH to increase due to the hydration of OPC and the pH buffering effect by the dissolution of solid phase SiO_2 contained in the silica fume.

(c) Analysis results

A dissolution test was carried out for silica fume to acquire data for deriving a dissolution rate equation (Iriya et al., 2004a); the dissolution rate equation to be used in the calculation model was derived based on the data (Yoshida and Mihara, 2005). Figure 2.2.6-12 shows the test result and the derived dissolution equation for the silica fume.

Using the resulting equation and considering the dissolution-precipitation reaction of the cement hydration products, the temporal evolution of pH was calculated based on the results of HFSC powder immersion tests as shown in Figure 2.2.6-10. In order to take into account the equilibrium reaction of the cement hydration products in this calculation in addition to the dissolution rate equation of the silica fume, the generation of CSH gel was calculated based on the equilibrium reaction of solid solution (Yoshida and Mihara, 2005). The result is shown in Figure 2.2.6-13. In the figure, the data from the immersion tests are plotted and the result assuming the formation of CSH gel is presented as a solid line and assuming no formation of the CSH gel as a dashed line. The calculation model is based on equilibrium reaction of the hydration product of cement and silica fume dissolution rate.



$$\text{Total Si} = t \times A \times 10^{-16.2} \times (a_{H+})^{-0.5} \times (1 - Q/K)^{4.3}$$

Here,

- t: Time
- A: Surface area
- a_{H+}: Hydrogen ion activity
- Q: $H_4SiO_4(aq)$ activity in liquid solution
- K: H4SiO4(aq) activity obtained by solubility product constant of silica fume

Figure 2.2.6-12 Results of elution test of silica fume and its kinetic equation

In the calculation assuming the formation of CSH gel, the pH decreases drastically after around 200 days and reaches a value of 10.3. This is because the dissolution of the silica fume supplies Si that consumes Ca to produce CSH gel and the solid phases that contribute to raising pH, such as portlandite ($Ca(OH)_2$), are rapidly depleted. In the calculation assuming no formation of CSH gel, the pH decreases to 11.9 in around 100 days and reaches a steady state. This is because, contrary to the calculation assuming the formation of CSH gel, the reaction of hydrate products that increases the pH remains dominant. The calculation assuming the formation of CSH gel did not reproduce the slow decrease in pH observed in the tests, which suggests that the generation of the CSH gel is the main factor in lowering the increase in pH.

The calculation assuming the formation of CSH gel reproduces the test results in the initial stage of the reaction, but not the slow decrease observed in the tests after the pH reached 11.5. On the other hand, the calculated values show a drastic decrease from around pH 11.5 due to the reduction of the Ca/Si ratio in the CSH gel. The calculation may therefore have to include other factors, including CSH gel formation reaction rate, when the Ca/Si ratio is low. The change with time in the CSH gel formation rate is not known for the time being and further studies will be required.

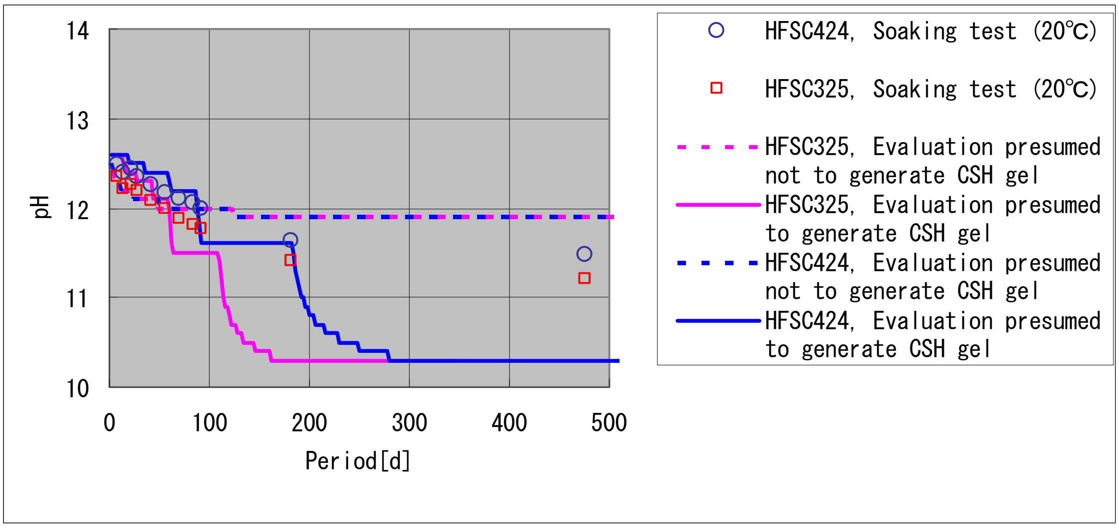


Figure 2.2.6-13 Comparison between analytical results and experiments in hydration of HFSC

However, the formation of CSH gel has been observed in a system where the hydrate products of cement are present and the pH of the actual immersion liquid should therefore come close to the calculated values assuming the formation of CSH gel with time.

Table 2.2.6-3 provides a summary of the various HFSC performance tests conducted to date.

Table 2.2.6-3 Overview of various performance test with HFSC
(○: implemented item)

	Item	OPC	HFSC			Outline
			424	325	226	
Confirmation of workability	Manufacturing test for segment used high-flow concrete				○	Casting performance was confirmed to manufacture segment with high-flow HFSC226 concrete.
	Preliminary shotcreteting test	○	○		○	HFSC424 shotcrete had the same construction performance as OPC shotcrete by preliminary test, ensured the design strength for Horonobe URL.
Grasp of quality variation	Slump flow test		○	○		It was confirmed to be able to manage quality change of fly ash sampled from the same thermal power plant every month.
	Compressive strength test		○	○		HFSC424 and HFSC325 almost ensured the design strength of cast-in-place concrete for Horonobe URL. The influence that the quality change of fly ash had on compression strength is small.
Estimation of durability	Exposure test of reinforced concrete under marine condition	○			○	As for HFSC226, the progress of corrosion was low though the corrosion beginning is earlier than OPC.
Accomplishing target pH	Soaking test used powdered specimen	○	○	○	○	When HFSC226 was soaked in 60days at 65°C, pH decreased to 11 or less, though pH didn't decrease to 11 or less even if HFSC424 was soaked at normal temperature for 456 days.
	pH analysis for soaking test		○	○		When the generation of the CSH gel was assumed by the calculation that used elution kinetic equation delivered from dissolution test of silica fume, pH decreased up to 10.3.

2.3 Confirming the applicability of the H12 Report design method for the surface-based investigation phase of the Horonobe URL project

2.3.1 Confirming the applicability of H12 Report design method

In the H12 Report, data on rock properties required for the design were collected, their trends and correlations were estimated and a relevant geological environment was assumed, based on literature surveys on a wide range of geological environments in Japan. Design studies were conducted for the engineered barriers and the disposal facilities at a disposal depth that was determined considering currently available construction technologies and long-term safety requirements. The design concept and method used in the H12 Report were applied to the design of the engineered barriers and the repository for the geological environment conditions in Horonobe as an example of actual environment. The points that were controversial in this study and recommended methods are summarized below.

(1) Overall design flow of the repository

A problem was identified regarding the overall design flow for the repository during the studies on individual design flows for the overpack, buffer and backfill, namely that the correlation among the individual designs, including facility design, was not clear. The overall design flow for the repository presented in the H12 Report was therefore revised to improve the generic nature of the flow by clarifying the correlation among the design factors.

(2) Evaluation of mechanical stability of tunnels

(a) The following two points need to be taken into consideration when evaluating tunnel stability and designing the support for a single tunnel:

- The design of supports should not be based on theoretical analyses as shown in the H12 Report, but on the tunnel stability analysis based on standard support patterns developed by empirical methods.
- The evaluation indicators used in the H12 Report are not realistic for quantitative evaluations. Indicators that take into account the concept of intelligent construction, which is becoming increasingly common for construction of underground vaults, would be required to function over the long-term that includes design, construction, operation and closure.

(b) The siliceous mudstones of the Wakkanai Formation found at the Horonobe URL site are sedimentary rocks with low strength. They also have features not considered in the H12 Report, i.e. anisotropic pressure and porosity. Therefore, when evaluating tunnel stability or designing support for a similar environment, the following points would have to be considered in addition to the elastic-perfectly plastic analysis conducted in the H12 Report:

- Long-term stability of rock under anisotropic stress
- Long-term deformation of rock during the construction and operation periods
- Hydro-mechanical coupled phenomena immediately after excavation, which is a crucial factor for porous rock
- Method for determining values for the analysis taking into account the anisotropy

(c) A disposal pit located deep underground in sedimentary rock with a low competence factor may remain stable without support for a short period of time, but would become unstable due to creep of the rock before the emplacement of waste packages. Therefore, when evaluating the stability of disposal pits (without support), creep behavior during the construction and operation periods should be taken account in addition to the elastic-perfectly plastic analysis.

(d) Determining disposal tunnel spacing and waste package emplacement pitch are design features that are unique to geological disposal. Experience in conventional underground facilities cannot provide any input for this and it is therefore necessary to design with a greater safety margin than for single tunnels. In other words, a comprehensive analysis of tunnel stability would be required, including complex destruction mechanisms of rock (strain softening and creep behavior) in addition to the evaluations conducted in the H12 Report. For the reinforcement of connecting zones of tunnels, a three-dimensional plastic analysis was carried out for the tunnel geometries and layout in the Horonobe URL design to determine the extent of influences and a simulation was then made in which stress was re-allocated three-dimensionally onto existing support in order to check its integrity, as in the H12 Report.

As described above, some new design factors and evaluation indicators were proposed based on the design methods adopted in the Horonobe URL. Applying these results to the repository design will require additional consideration of safety performance, as described later in section 2.4.2.

(3) Design of the engineered barriers

The concept in the H12 Report would be applicable for the design of the engineered barriers. Particularly for the buffer, flexible design is possible by defining specifications that meet the requirements with a certain range depending on the geological conditions of interest. A detailed design flow for defining the basic specifications of the buffer was not presented in the H12 report, but a method could be proposed in the present study. The data to be accumulated were also clarified through the case study design. For evaluating the long-term integrity of the engineered barriers, the reliability of the method was improved by improving models and increasing input parameters since the H12 report. Its applicability was also confirmed in this study.

(4) Design of closure

For this topic, a design flow was developed and the flow was applied in a design using actual geological environment conditions as an example. Applicability of the evaluation method from the H12 Report was basically confirmed.

2.3.2 Points concerning the surface-based investigation phase of the Horonobe URL project

Important points are listed in terms of investigation method of the geological environment, design method, material selection, data setting and safety function limitations, based on the discussions in sections 2.1 and 2.2. They are grouped into the following five categories: (1) investigation of the geological environment, (2) design of the facilities, (3) design of the engineered barriers, (4) design of the closure system and (5) others.

(1) Investigation of geological environment

- If concrete support is not used in construction of the disposal pit, it will be necessary to evaluate whether the geological conditions would ensure that the disposal pit would be mechanically stable without support.
- When only limited materials can be used for support in the disposal facilities due to the chemical stability of the concrete, it will be necessary to evaluate whether the geological environment will ensure tunnel stability with support using the selected materials.
- Hydrochemical data may be affected by contact of samples with the atmosphere. Therefore, it is important to obtain a comprehensive understanding of the depth dependence of groundwater chemistry, results of thermodynamic analyses of groundwater chemistry and information on minerals contained in the formations of interest.

(2) Design of facilities (tunnel stability, support design and disposal tunnel spacing)

- Materials for supports and grouting should be developed considering their chemical influence on the near-field environment and restrictions on their application should be studied. Based on the result, appropriate materials should be selected and their characteristics taken into account in design and construction.
- Unlike other underground facilities, the repository has to be designed with consideration of its containment function. The design should include consideration of safety from the viewpoint of both construction and safety assessment.
- Surface-based investigations do not provide sufficient information to clarify in detail the range of conditions in the deep geological environment where waste packages will be emplaced. This type of knowledge has to be acquired using a stepwise approach. A design based on surface-based investigations should be constantly updated through the intelligent construction process and design and construction methods for support measures should be selected based on information obtained during construction. In the design phase, analyses should focus on parameters that can be compared with data obtained during construction and operation (e.g. convergence) It is also desirable to include margins in the values used for the analyses in the design phase.
- The design of disposal facilities requires information on the specifications of the engineered barriers, such as geometry and dimensions. In turn, the design of the engineered barriers and the backfill requires information on facility specifications, such as waste package emplacement pitch and thickness of support. Unlike the concept in the H12 Report, in which the facility, the engineered barriers and the backfill were designed individually, it will be necessary to evaluate the correlations among these components once a disposal site has been selected and the influence of the geological environment on the individual design requirements has been studied.
- Although the double-support concept allows use of supports with reduced thickness, the concept still requires further studies on safety aspects, due to the fact that it allows rock loosening to a certain extent.
- Borehole investigations in the vicinity of the planned access tunnel would minimize uncertainties and risks during the construction phase.
- It is known from experience that, in general, structures deep underground are less susceptible to impact from earthquakes. However, there are no guidelines or standards regarding seismic resistance for deep underground facilities and the influence of earthquakes also depends on geological conditions. The influence of earthquakes will have to be evaluated for each geological environment of interest.

(3) Design of the engineered barriers (including an assessment of long-term integrity)

- In the case study design of the buffer, the swelling pressure of 0.1 MPa to be reached at after self-sealing of the gap around the buffer, which is the criterion used by Posiva (Dixon, 2000), was used as the basis for determining the self-sealing properties of the buffer. Its validity should be checked, particularly for saline groundwater conditions, through experiments considering more realistic gaps or groundwater infiltration.
- The applicability of the method for evaluating the long-term integrity of the engineered barriers was tested for evaluating the long-term mechanical deformation of rock and buffer, extrusion of buffer into fractures and gas migration through the buffer. Regarding extrusion of buffer into fractures, test results indicate that the rate of the intrusion into fractures under saline groundwater conditions falls in a range between 1/4 and 1/10 of that for freshwater conditions.
- When designing the engineered barriers, the specifications of the buffer should be defined in a range that would not cause passivation of steel. When designing the overpack, buffer and backfill, the interactions among these components should be evaluated and the focus should be on factors with greater influences, allowing the design process to be rationalized.

(4) Design of the closure system

- The performance of the backfill can be influenced by groundwater, depending on its ionic strength. When concrete is used as a support material, the quality of the concrete can be degraded, resulting in preferential flowpaths for the groundwater, which might have a significant influence on the overall performance of the repository. In order to design a closure system that can compensate for such influences, a basic design concept for the system as a whole, as well as for the individual components such as sealing plugs and backfill, should be clarified. In addition, more data should be collected on the self-sealing properties of backfill with a low bentonite content under saline groundwater conditions.

(5) Others

- In the H12 Report, phenomena such as destabilization of tunnel face, generation of spring water and gas blowout were listed and construction measures to prevent them were described. When designing for a specific geological environment, the list should be expanded to include geology-specific phenomena as well as all potential disasters such as fire and countermeasures planned accordingly.

- For methane gas blowout, it should be estimated whether the blowout rate is within a range that can be controlled by ventilation, meaning that construction and operation would not be disturbed by gas blowout. If this is not the case, suitable countermeasures should be prepared.
- As a measure against spring water, sealing is preferable to drainage from the viewpoint of preserving the geological environment. In disposal facilities, however, care should be taken when selecting the materials for sealing plugs, as in the case of the support measures.
- Tunnel fire is the most likely accident during the construction and operation period. Simulations using a ventilation network analysis, for example, should be carried out to determine how fire could occur and the results should be reflected in the layout design. Suitable provisions for ventilation and evacuation have to be made.

2.3.3 Conclusions

For development of the overall design flow for a repository, complex relationships among the individual designs for the facilities, engineered barriers and backfill were studied in terms of their influence using the vertical emplacement concept as an example. The overall design flow shown in the H12 Report was revised based on the result in such a way that the design process could be simplified as much as possible. Major differences from the H12 Report include: (a) evaluation of tunnel stability is carried out before the design of the engineered barriers because this is required to determine the thickness of support for disposal tunnels and to determine the need for support for disposal pits, which, in turn, is important for defining the specifications of the backfill and buffer, (b) the design of the backfill is carried out before the design of the overpack because this has a large impact on determining the corrosion allowance for the overpack.

The applicability of the design methods based on the revised design flow was evaluated for the vertical emplacement concept under the geological conditions at the Horonobe URL, where siliceous mudstones of the Wakkanai Formation are found, as a case study for sedimentary rock. The disposal depth was assumed to be 450 m in order to meet the requirements that the mechanical stability of the tunnels should be assured and that the rock in the emplacement zone should have a sufficient thickness (the analysis model in the H12 Report suggests that the thickness should be approximately 5 times greater than the diameter of the disposal tunnel).

Input data for the design were determined as described below, based on information obtained from borehole investigations at the Horonobe URL site. Because the porosity of sedimentary rock such as that found at Horonobe varies with depth, input data on density and thermal properties were determined using a correlation between depth and the data obtained from the borehole investigations. For the data on mechanical properties, a model was developed for each rock mass classification; the data obtained from the borehole investigations were adjusted according to these classifications and then assigned. Since the initial stress ratio obtained from the investigations was almost constant regardless of depth, the average value was used as input. For the hydrological properties, input data on hydraulic conductivity were determined based on the results of permeability tests in the zone around the depth of interest and data on the hydraulic gradient were obtained by a groundwater flow analysis. For groundwater chemistry, input data were determined based on information on groundwater sampled from boreholes, as well as on a comprehensive analysis of depth-dependent properties, the results of thermodynamic analyses and information on the minerals found at the URL site.

For facility design, the stability of disposal pits and tunnels and the waste package emplacement pitch were evaluated for the geological environment at the Horonobe 450 m level according to the revised design flow, in the same level of detail as in the H12 Report. Some additional design items were also identified in the course of the study using actual geological environment data. Through these studies, the design methods used in the H12 Report were evaluated in terms of their applicability, and points to be improved for the design were identified.

Regarding the design of the engineered barriers, the principles of the design procedure for the buffer and overpack were formulated and individual design flows were developed. The design flows were applied to the case study design for the buffer and overpack under the geological conditions at Horonobe. As a result, the specifications in the H12 Report were confirmed as being appropriate. The long-term integrity of the engineered barrier system with the specifications discussed above was evaluated in terms of long-term mechanical deformation of the rock and buffer, extrusion of buffer into fractures and gas migration through the buffer. It was confirmed that engineered barriers with the specifications defined in the H12 Report would, as an overall system, maintain long-term integrity. Some issues to be addressed in the future identified through these studies were also summarized.

With regard to the design of the closure system, design requirements and geological environment conditions for the backfill and sealing plugs were summarized and a preliminary design flow was developed. Some examples of material specifications for the closure system were defined for the geological environment at Horonobe based on the result of property tests and R&D being carried out in other countries. Issues to be addressed in the future were also summarized.

For the low alkaline concrete (HFSC), prototype segments were manufactured and shotecreting tests were conducted in a simulated tunnel to evaluate workability at emplacement. It was confirmed that the design-based strength required for the Horonobe URL could be achieved by this type of concrete. It was also confirmed that variability in fly ash quality and its influence on the strength of fresh concrete would fall within a range that could be adjusted with high-performance AE water-reducing admixture. To evaluate corrosion of rebars in the concrete, exposure tests were conducted in the splash zone above the sea. The result for up to three years indicates that, although corrosion occurred earlier in HFSC226 concrete than in OPC concrete, the propagation rate of corrosion is smaller in the third year than in the first year. The tests will be continued. To investigate the decrease in pH, immersion tests were conducted using concrete paste powder. The pH of HFSC424 at room temperature did not decrease to 11 even after 456 days, while that of HFSC226 immersed at a temperature of 65°C decreased to 11 or lower after 60 days, i.e. the decrease in pH is temperature-dependent. This indicates that increasing the pozzolan content and the curing temperature would be advantageous in decreasing the pH of the concrete. A dissolution rate formula was derived for silica fume from the results of dissolution tests. An analysis was made using this formula to reproduce immersion test results at room temperature, assuming the formation of CSH gel and consumption of Ca. The result of the analysis suggests that the pH decreases to around 10.3, which means that the test result was reproduced sufficiently well. A more precise model should be formulated for analysis of CSH gel dissolution and precipitation. It cannot be concluded from these results that the effectiveness of HFSC was validated as a low alkaline concrete. However, some issues to be addressed in the future were identified, such as the influence of high temperature on pH decrease due to accelerated pozzolanic reaction whereby the pH decrease should be analyzed considering pozzolanic reaction rates at low a Ca/Si ratio.

2.3.4 Future work

Issues to be addressed in the future identified through studies on the applicability of a range of disposal technologies under the geological environment conditions at Horonobe are summarized below.

(1) Overall design flow for the repository

An overall design flow for the repository for horizontal emplacement needs to be developed, because the design flow was revised for the vertical emplacement concept in this study and the correlation factors among individual design items will be different for the vertical and horizontal emplacement concepts. The overall design flow should be refined to allow a more practical design flow based on the construction phase of the Horonobe URL project and by validating the methodology used in this study.

(2) Determination of design data

The facility design method, methods to characterize the geological environment and advertency points revised from the H12 Report based on the information from the surface-based investigations should be demonstrated using information obtained during the construction and in-situ test phases of the URL project. These methods should be further improved by incorporating the additional information so that more rational and reliable design methods could be established.

(3) Facility design (tunnel stability, support design and disposal tunnel spacing)

Evaluation of the stability of disposal pits in sedimentary rock requires analyses that take rock creep during the period between excavation and emplacement into account. Details of the design for the disposal pit should be studied considering use of low-alkaline concrete support, although, in principle, the disposal pits have been designed without any support so far.

The H12 Report and the present study used an elasto-plastic analysis and evaluation indicators to determine disposal tunnel spacing. Parallel construction of as many as 50 tunnels in a single panel would be unique to geological disposal and no relevant experience is available from construction of conventional underground facilities. Inappropriate design could lead to disasters such as collapse of all the tunnels. The reliability of these design methods should therefore be further evaluated. For example, consideration of strain softening and rock creep in addition to the elasto-plastic analysis would be required.

There are various restrictions on the facility design, not only in terms of ensuring the mechanical stability of the tunnels, but also for ensuring the required performance of the engineered barriers and the near-field geosphere. It is important to identify all these restrictions and to reflect them in the design and construction. For example, it is assumed that the rock around the tunnels and the concrete supports will deteriorate over long time periods, resulting in an increase in permeability. Detailed clarification of such phenomena is difficult and provision of appropriate measures is very important for ensuring system safety in the event that such phenomena occur. To be more specific, studies should be made on the self-sealing properties of the rock, swelling of the backfill and the characteristics and applicability of measures such as sealing plugs and grouting. An approach for evaluating the EDZ considering containment capability should be formulated, which should then be reflected in the design. The support materials that can be used in the repository may be restricted due to potential chemical influences on the near-field environment. Further research and development should be carried out on, for example, OPC and low alkaline concrete as candidate materials for support and grouting, in order to understand their applicability and restrictions. Suitable materials should then be selected and design and construction should be implemented according to the properties of the selected materials. A considerable hydrostatic pressure applies to facilities located deep underground. Different design and construction concepts can be used to deal with this pressure, including construction of conduits to reduce the water pressure on the support, as seen at the Horonobe URL site, ground improvement by grouting or applying the freezing method to reduce hydraulic pressure and installing segments or other rigid supports to resist the pressure. Since safety performance is a requisite for the disposal project, the most suitable solution seems to be one in which water is discharged through a tunnel during the operation period, which is then closed at the time of backfilling. More detailed studies are required, focusing on the safety aspects of the design methods.

(4) Design of the engineered barriers (including evaluation of long-term integrity)

One of the design requirements of the buffer is to have a self-sealing function. The criteria adopted by POSIVA (Dixon, 2000) whereby a swelling pressure of 0.1 MPa should be reached after self-sealing of the gap around the buffer, should be demonstrated by collecting more data with gap width as a parameter, particularly under saline groundwater conditions. The applicability of the in-situ compacting method for buffer in a sedimentary rock environment should also be evaluated in terms of its influence on the rock.

The thickness of the overpack should be determined as the sum of the corrosion allowance and the thickness required for radiation shielding, in so far as the latter is not less than the thickness required to withstand external loading. Since radiation from HLW glass decreases with time, a rational design concept should be developed considering possible relationships between the corrosion allowance and the radiation shielding thicknesses. An experimental study should be carried out on corrosion behavior in realistic groundwater conditions (including the influence of support materials), and on radiation effects that have a large impact on determining the thickness, in order to obtain data on alternative overpack materials.

(5) Design of the closure system

The performance of the backfill may be influenced by groundwater depending on its ionic strength. In order to take such an influence into account in the design of the closure system, the basic design concept for the system as a whole and the individual role of each closure component, such as sealing plugs and backfill, should be clarified. More data should be collected on self-sealing properties of backfill with a low bentonite content under saline groundwater conditions. The closure components should also be evaluated in terms of potential formation of groundwater flowpaths due to deterioration of the support. Information on the EDZ should be collected during the construction of the URL and reflected in the backfill design.

(6) Development of low alkaline concrete

The properties of HFSC concrete, such as fluidity and strength, vary depending on the type of pozzolanic materials, their composition, environmental conditions and elapsed time. Properties such as workability, variability of quality, temporal changes in strength and the decrease in pH due to the pozzolanic reaction should be evaluated by demonstration tests in an experiment drift at the Horonobe URL. Modeling the decrease in pH requires intensive collection of data on parameters relevant to the reaction rate. The mechanism of CSH gel formation and parameters relevant to the formation rate should be investigated to obtain a more precise analysis of the decrease in pH of the water penetrating through concrete structures.

2.4 Applicability of engineering technology to the construction phase of the Horonobe URL project

JAEA is pursuing geoscientific research on sedimentary rocks at Horonobe in Hokkaido. Construction of the URL started in 2005 and the project has transitioned from Phase I, focusing on surface-based investigations, to Phase II, which involves investigations during tunnel excavation.

Applicability of the engineering technologies presented in Supporting Report 2 of the H12 project to a specific geological environment has been evaluated based on the information obtained in Phase I of the Horonobe URL project, as part of the development of fundamental engineering technologies. "Engineering technology" here means all the technologies required for the design of the engineered barriers and the underground facilities, including technologies related to construction, operation and closure, design methods based on geological information obtained stepwise, data required for the design (including data for the evaluation of long-term behavior) and models based on an understanding of geological phenomena (knowledge of the long-term behavior of the engineered barrier system and the geosphere).

Information accumulated with the progress of tests and investigations in Phase II of the project should be used to evaluate the applicability of engineering technologies and to improve their reliability. The results should be compiled systematically and reflected in the knowledge base.

The program for Phase III will include detailed investigations of the geological environment around the URL and various in-situ tests for R&D on geological disposal.

The following descriptions of test programs, starting times, test locations and test periods for investigations of the applicability of engineering technologies in Phase II will be subject to change depending on progress of the construction of the URL and other circumstances.

2.4.1 Experiment plan for the construction phase of the Horonobe URL project

Figure 2.4.1-1 shows the facility layout planned for the Horonobe URL. Based on progress as of the end of 2005, three vertical shafts will reach 500 m depth and the first ring tunnel (at around 400 m depth) will be constructed by around 2013 (Matsui, 2005).

In Phase II of the project, investigations are planned with the aim of validating geological environment models, rock mechanical models and support designs developed in Phase I, and clarifying the mechanisms of the EDZ and formation of an unsaturated zone, i.e. factors that were difficult to investigate in Phase I. The applicability of low alkaline concrete as a construction material will also be evaluated in order to improve the reliability of disposal technologies and gas migration (Matsui, 2005). The Phase II program also includes selection of the site and planning of the program for in-situ tests to be conducted in Phase III. Table 2.4.1-1 shows the items for these investigations.

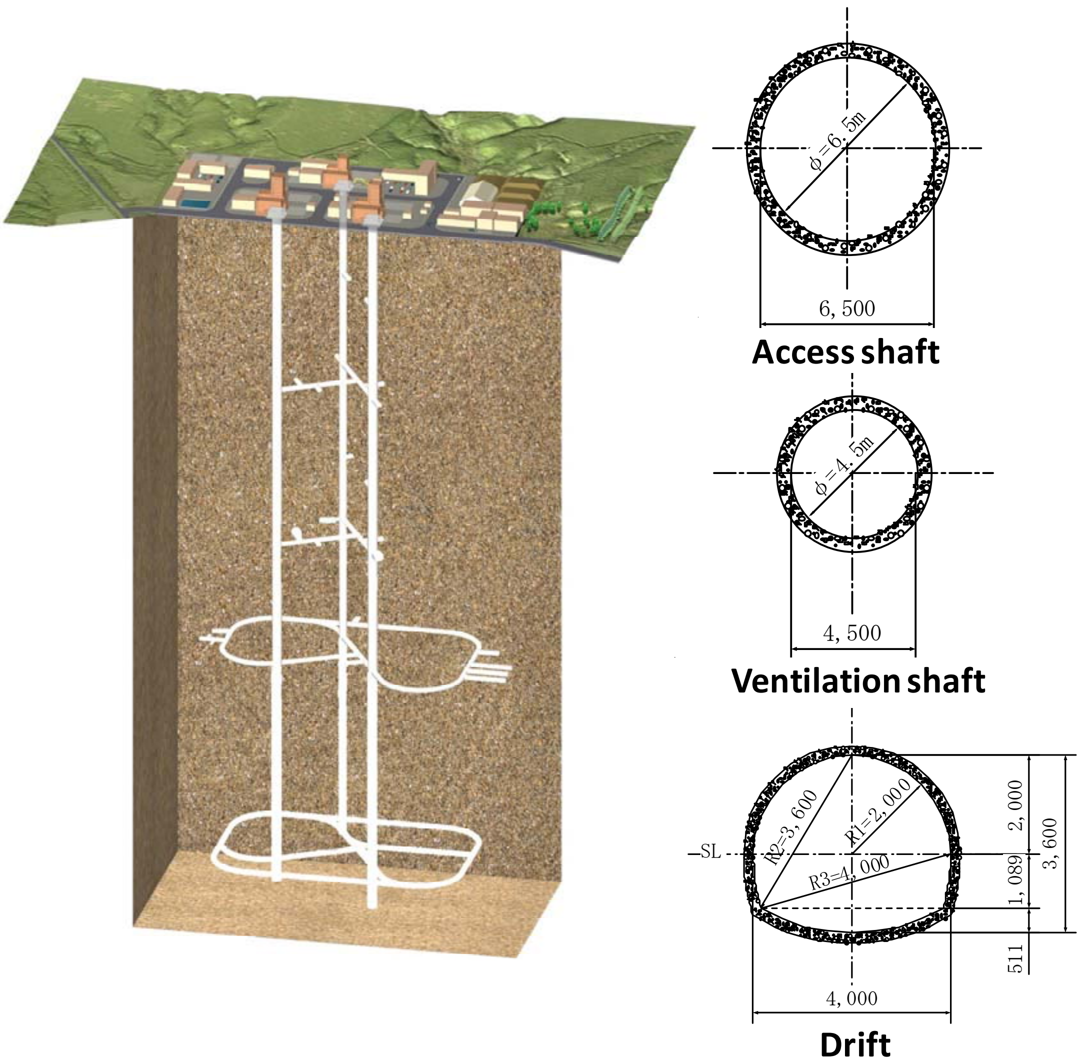


Figure 2.4.1-1 Image of the URL facilities

Table2.4.1-1 Horonobe underground research project
plans of investigation and examination at second phase (Aoyagi et al.,2006)

Classification / Purpose	Item of investigation examination
1. Geoscience study	
(1)Investigation examination to carry out from a shaft	
Propriety studies of geologic and geological structure model that built at the first phase	①Geological observation
Propriety studies of hydrogeological structure model that built at the first phase	②Measurement of spring quantity in shaft
	③Pressure distribution investigation of groundwater of rock around tunnel
	④Water quality distribution investigation of groundwater of rock around tunnel
Propriety studies of rock mechanical structure model that built at the first phase	⑤Investigation about properties of EDZ of rock around shaft
	⑥Measurement of deformation behavior of rock around shaft
Propriety studies of support design that carried out at the first phase	⑦Measurement of support stress
	⑧Deformation measure of tunnel support (steel support)
(2)Excavation influence examination for rock around shaft that is carried out at horizontal tunnel	
Impact statement to rock by excavating, Propriety studies of mechanical model of rock, Elucidation of outbreak mechanism of EDZ	①Stress distribution and measurement of stress change of rock around tunnel
	②Measurement of change of dynamics properties of rock around tunnel
	②Measurement of change of water permeability of rock around tunnel
(3)Investigation around shaft／horizontal tunnel	
Propriety studies of Geological feature environment model that built at the first phase, Evaluation of heterogeneousness of geological environment model for the third phase study	①Investigation about geological structure
	②Investigation about hydrological, geochemical, rock dynamics
(4)Investigation and examination that is carried out at horizontal tunnel	
Grasp of outbreak mechanism of EDZ, Outbreak area of EDZ and the acquisition of properties	①Examination of excavation influence at horizontal tunnel
Grasp of outbreak mechanism and area of undersaturation, Grasp of area of chemical composition subsurface water change domain	②Investigation and examination of undersaturation and REDOX
2. Dependability enhancement of disposal technology	
(1)Investigation and examination that is carried out at horizontal tunnel	
Execution confirmation and coverage of ground supports, Execution confirmation such as plug, grout	①Confirmation test of execution of low alkaline concrete
Proof of functions of whole system, Propriety studies of migration evaluation model of gas generated with corrosion of overpack	②Behavior test of gas migration

2.4.2 Development of engineering technology in the construction phase of Horonobe URL project

This section describes the investigations development of related technologies to be conducted in Phase II of the Horonobe URL project.

As shown in Table 2.4.2-1, development of engineering technologies is divided into three categories: (A) tests and investigations using information to be obtained during the construction of the URL or using the URL itself, (B) laboratory tests and analyses that are not directly related to the construction of the URL, but involve the geological conditions at the site (e.g. geological structure, hydrology, geochemistry and rock mechanical conditions) (see section 4 of the separate volume on geoscientific research) and (C) research requiring neither information on the geological conditions at the site nor validation at the URL (Aoyagi et al., 2006).

Table2.4.2-1 Classification of item of research and development about engineering (Aoyagi et al.,2006)

Research and development about engineering	Classification
Various information to be provided with construction of Horonobe URL and research using URL in itself	A
Items that construction of Horonobe URL does not have direct relations and indoor examination and analysis to carry out for geological environment condition of Horonobe	B
Studies that does not carry out inspection in URL without intending for geological environment condition of Horonobe	C

Table 2.4.2-2 shows individual research items. These were listed based on the classification of research items in the field of disposal technology contained in the report entitled "Overall R&D map for geological disposal of HLW" of RWMC (2002). This table includes both the classification from Table 2.4.2-1 for development items and that from Table 2.4.2-3 for in-situ test items to allow their relationships to the individual research items to be clearly understood.

Table2.4.2-2 Individual study item about engineering (Aoyagi et al.,2006)

Individual item	Classification (Table2.4.2-1)	Correspondence with in-situ test item (Table2.4.2-3)
(1) Engineering		
① The whole		Master plan, Unification of individual item, Base technical systematic maintenance, Examination of disposal general idea and design option
② Artificial barrier		
1) Basic characteristic of overpack	B	d
2) Basic characteristic of buffer material	B	a, b, e
③ Supprt, Grout, Ceiling		
1) Ceiling	B	h
2) Low alkaline cement	A	f
3) Grout	A	e
④ Construction, Operation, Closure		
1) Construction technology	A	all (designs such as test gallery)
2) Operation technique	B	g
3) Decommissioning technology	B	h
4) Quality management	A	all (b in particular)
5) Retrieval technique	C	×
(2) Long-term soundness		
① Glass solidification	C	×
②Buffer material		
1) Long-term dynamic deformation behavior of buffer material	B	b
2) Long-term transformation behavior of buffer material	B	a, d, e
3) Behavior of outflow and invasion of buffer material	B	×
③Cement･Concrete	C	×
④Rock	A	b, e
⑤Coupled evaluation technology of T-H-M-C	B	a
⑥Migration behavior of gas	B	c
⑦Behavior of shear reply of buffer material	C	×

"(1) General" shows the general items related to R&D on engineering technologies.

"(1) (c) 2 Low alkaline cement," "(1) (c) 3 Grout," "(1) (d) 1 Construction technology," "(1) (d) 4 Quality control" and "(2) (d) Rock" are R&D items investigated using information obtained during the construction of the URL and using the URL facility, classified as (A) above.

"(1) (d) 5 Retrieval technologies," "(2) (a) vitrified waste," "(2) (c) Cement and concrete" and "(2) (g) Buffer response to shearing" are not R&D items related to understanding geological environment in Horonobe and not intended to be validated in the Horonobe URL project, thus classified as C.

Items other than those above include laboratory tests and analyses that are not directly related to the construction of the Horonobe URL, but involve the geological conditions at the site or preparation for the in-situ tests in Phase III, thus classified as (B).

Table 2.4.2-3 shows the in-situ tests on engineering technologies planned in the latest Horonobe URL program, summarized on the basis of reports by Matsui (2005) and Kurihara et al. (2004).

Table2.4.2-3 In-situ test item for engineering technology
(Aoyagi et al., revised 2006 partly)

In-situ test item	Classification (Table2.4.2-1)	Study item (Table2.4.2-2)	Enforcement stage
a. T-H-M-C coupled test	B	(1)②2), (2)②2), ⑤	3 phase
b. Creep test of buffer and rock	B	(1)②2), (2)②1), ④	3 phase
c. Gas migration test	B	(2)⑥	2～3 phase
d. Corrosion test of overpack	B	(1)②1), (2)②2)	
e. Influence test of cement	B	(1)②2) , ③2) , ③3) , (2)②2), ④	3 phase
f. Confirmation test of execution of low alkaline concrete	A	(1)③2)	2～3 phase
g. Confirmation test of precision of setting	B	(1)④2), 4)	3 phase
h. Closedown test of gallery	B	(1)④3)	3 phase
i. Grout	A	(1)③2), (1)③3)	2～3 phase

Low alkaline concrete construction performance tests and gas migration tests are planned to start in Phase II (Matsui, 2005). However, the gas migration tests should be started earlier to ensure a longer saturation time for the buffer in the experiment drift. The low alkaline concrete construction performance tests will be the only tests actually related to the construction work of the URL.

2.4.3 Approach to studying individual items

Of the individual research items shown in Table 2.4.2-2, the key items and a general description of the activities in Phase II are outlined below. The items classified as (C) were omitted because they are not related to the Horonobe project.

(1) Category A (tests and investigations using information to be obtained during the construction of the Horonobe URL or using the URL itself)

1) Low alkaline cement

Shotcreting and lining concrete techniques using low alkaline cements such as HFSC will be tested in part of the horizontal drift at the URL to demonstrate the construction of low alkaline concrete at intermediate depth and to identify issues to be addressed. The long-term integrity of the cement and concrete will be evaluated based on R&D results from other institutes in Japan (The Federation of Electric Power Companies and JAEA, 2007) and from foreign URLs.

2) Grouting

Grouting techniques for sealing inflow during the construction and operation phases will be evaluated and will be applied in the Horonobe URL to demonstrate applicability to an actual geological environment. More specifically, the grouted zone will be monitored and its pH and Eh evaluated in terms of influence of the cement on the rock (core sampling) as part of cement impact tests. Workability tests will also be carried out for grouting materials where there is less experience with application, such as low alkaline cement.

3) Construction technology

In Phase II, the applicability of the design methods used in Phase I for access shafts and single tunnels will be evaluated using in-situ data obtained through measurements and investigations during construction of the URL. Efforts will also continue to obtain more detailed property values for the design and to establish a method for designing a many tunnels deep underground.

An intelligent construction plan will be formulated for the construction of the URL and will be demonstrated for construction in Phase II (mainly access shafts) as a case study.

Based on the experience gained during the construction of the URL, construction technologies for underground facilities will be evaluated. In this process, requirements for construction technologies and safety assessment from the view point of emplacement technology will be identified.

4) Long-term stability of rock

In order to improve the reliability of long-term predictions, models used to predict the rock behavior around the tunnels will be evaluated by comparing model predictions with actual data obtained during excavation in Phase II. Additionally, a method to determine constants to be used in the models, which was developed based on the surface-based investigation in the Horonobe URL site, will also be validated. For the buffer/ rock creep tests planned in Phase III, more data on mechanical properties will be acquired using rock samples obtained in the vicinity of the planned in-situ test site using a long borehole drilled from the underground facilities. A method will also be developed for analyzing complex behavior in the near-field. In cooperation with other task groups responsible for evaluating system performance and investigating the geological environment, and through tests of excavation impact in Phase II, the EDZ and its long-term evolution will be characterized, self-sealing properties (strength and permeability) will be evaluated and more detailed EDZ models developed for the safety assessment.

The long-term alteration of near-field rock will be evaluated for the case where OPC is used as rock support and grouting material. Specifically, using existing results from Japan (The Federation of Electric Power Companies and JAEA, 2007) as a basis and also results from the Long-term Cement Study (LCS) which is a collaborative research project with NAGRA at the Grimsel Test Site, a cement influence test will be conducted using grouting and concrete supports in the Horonobe underground facility.

(2) Classification B (laboratory tests and analyses that are not directly related to construction of the Horonobe URL, but involve the geological conditions at the site (geological structure, hydrology, geochemistry and rock mechanical conditions))

1) Basic properties of the overpack

The applicability and corrosion lifetime of candidate materials for the overpack will be evaluated for the conditions of the geological environment at Horonobe. A corrosion monitoring method under Horonobe groundwater conditions will be developed.

2) Basic properties of the buffer

The basic information required for design and material selection will be accumulated for the geological conditions at Horonobe, taking into account gap-filling capability of the buffer under saline groundwater conditions.

Data on the basic properties of the buffer will be acquired, focusing on the influence of cement and the coupled effects of saline water and cement, a method for evaluating these influences will be developed; this should contribute to optimizing the design and material selection for the buffer.

3) Long-term mechanical deformation of the buffer

A method will be developed for a coupled analysis of the long-term mechanical behavior of buffer and rock and in-situ test programs will be formulated.

4) Long-term alteration of the buffer

In cooperation with the R&D on disposal of TRU waste, a model and associated database will be developed to analyze the influence of cementitious materials on the buffer. Data required for verification of the model developed based on laboratory tests will be accumulated through large-scale tests in the actual geological environment.

5) T-M-H-C coupled analysis

As part of the development of analysis codes for coupled thermal-hydraulic-mechanical-chemical processes, experiments will be carried out using the geological environment data obtained in Phase II (for intermediate depth) in order to improve the reliability of the codes.

Coupled analysis codes using geological data obtained in Phase II will be used in the preliminary analyses of engineered barrier tests. The results will be reflected in determining the period and scale of the tests, which, in turn, will define the specifications of engineered barriers used in the tests, and in formulating the monitoring program. Together with the Agency for Natural Resources and Energy, the long-term reliability of technologies for monitoring the thermal and hydraulic behavior of the engineered barriers and their surroundings will be evaluated in order to support the formulation of monitoring program.

2.4.4 Future work

In this section, specific research items were summarized that represent the minimum requirement for Phase II of the Horonobe URL project

Progress in individual research areas should be reported on a regular basis and, based on the progress of construction of the URL and the latest trend in HLW geological disposal technology, individual research programs and in-situ test programs should be refined.

As a core R&D organization, JAEA is responsible for carrying out systematic and fundamental research. On the other hand, the remit of the Agency for Natural Resources and Energy of Japan is largely to promote engineering based technologies relevant for investigating the geological environment and relevant for repositories. JAEA and the Agency for Natural Resources and Energy of Japan are collaborating together to assimilate basic information related to repository safety measures and operation, and promoting research for developing fundamental investigative and engineering techniques etc. However there is still a need to develop an overall system that links the technologies used to investigate the geological environment, and technologies related to the repository design and quality management etc.

References

Adachi, T. and Oka, F. : “Constitutive equations for normally consolidated clay based on elasto-viscoplasticity”, Soil and Foundation, Vol.22, No.4, pp.57-70 (1982).

Aoyagi, S., Yui, M., Tanai, K., Kawakami, S., Fujita, T., Taniguchi, N., Shibata, M., Konishi, K., Nishimura, M., Kikuchi, H., Matsumoto, K., Matsui, H. and Nakayama, M. : “The Study Plan on Applicability to Actual Geological Environment Conditions of the Engineering Technology for Geological Disposal of High Level Nuclear-waste through the Second Phase in Horonobe Underground Research Project” (2006). (in Japanese)

ANDRA : "Dossier 2005 argile" (2005).

Architectural Institute of Japan : “Recommendation for the Design of Building Foundations” (2001). (in Japanese)

Autio, J., Riekkora, R. and Salo, J-P. : “Alternative backfilling concepts for a spent fuel repository at Olkiluoto”, Backfill Requirements in KBS-type repository -A POSIVA/SKB Workshop, Swedish Nuclear Fuel and Wastes Management Co. (SKB), IPR-02-05, pp.39-48 (2002).

Dixon, D.A. : “Porewatre salinity and the development of swelling pressure in bentonite-based buffer and backfill materials”, POSIVA 2000-04 (2000).

Fujita, H., Noguchi, S. and Hironaga, M. : “Development of Low Alkaline Cement Containing Häuyne -Effect of the Mixing of Silica Fume on the pH Behavior-”, Japan Atomic Energy Society, 1998 Fall Meeting, pp.823 (1998). (in Japanese)

Gray, M. and Shenton, B. : “Design and Development of Low-Heat, High-Performance, Reactive Powder Concrete”, High-Performance and Reactive Powder Concrete, Sherbrooke. PQ, Canada, 1998 August 16-20 (1998).

Greenfield, B.F., Ilett, D.J., Ito, M., McCrohon, R., Hearth, T.G., Tweed, C.J., Williams, S.J. and Yui, M. : “The Effect of Cement Additives on Radionuclide Solubilities”, Radiochim, Acta 82, pp.27-32 (1998).

Hirai, T., Tanai, K., Kikuchi, H., Shigeno, Y., Namikawa, T., Takaji, K. and Ohnuma, S. : “Study on Proper Constitutive Model for Evaluation of Long Term Mechanical Behavior of Buffer Material”, JNC TN8400 2003-034, (2004). (in Japanese)

Iriya, K., Mikami, T., Yasuoka, T. and Uegaki, Y. “Study on Construction Method of Concrete in the Underground Research Laboratory”, JNC TJ1400 2001-004, (2001). (in Japanese)

Iriya, K. and Mihara, M. : “Development of Lower Alkalinity Cement Containing Large Amounts of Pozzolanic Materials”, Proceeding of Japan Concrete Institute, Vol.25, No.1, pp.185-190 (2003). (in Japanese)

Iriya, K., Mikami, T., Takeda, N. and Akiyoshi, K. : “Study on Construction Method of Concrete in the Underground Research Laboratory (III)”, JNC TJ5400 2003-002, (2003a). (in Japanese)

Iriya, K., Tajima, T. and Noda, M. : “Study on Construction Method of Concrete in the Underground Research Laboratory (IV)”, JNC TJ5400 2003-008, (2003b). (in Japanese)

Iriya, K., Kurihara, Y. and Fujishima, A. : “A Properties of Low Alkaline Shotcrete using Large Amount of Pozzolans”, Japan Society of Civil Engineers, 59st Annual Meeting & Annual Conference, CS-1-29, pp.57-58 (2004a). (in Japanese)

Iriya, K., Kurihara, Y. and Noda, M. : “Private Communication” (2004b).

Ishijima, Y. and Kiyama, T. : “An Approach of the Long-term Stability of Strata by the Horonobe Research Institute for the Subsurface Environment”, Proceeding of 22th Annual Meeting of Atomic Energy Society of Japan Hokkaido Branch, pp.22 (2004). (in Japanese)

Japan Nuclear Cycle Development Institute (JNC): “Second Progress Report on Research and Development for the Geological Disposal of HLW in Japan H12: Project to Establish the Scientific and Technical Basis for HLW Disposal in Japan - Supporting Report 2 Repository Design and Engineering Technology -, JNC TN1410 2000-003 (1999).

Japan Railway Construction Public Corporation : “Guidelines for Design and Construction with NATM” (1996). (in Japanese)

Japan Society of Civil Engineers : “The Standard Specification of Tunnels -Mountain Tunneling Method- and Explanations” (1996). (in Japanese)

Japan Society of Civil Engineers : The standard Specification of Concrete Structures “Structural Performance Verification” (2002). (in Japanese)

Kawasaki, M., Taniguchi, N. and Kawakami, S. : “Anodic Polarization Behavior of Pure Copper in Carbonate Solutions and in Bentonite/sand Mixture”, JNC TN8400 2002-016 (2002). (in Japanese)

Kikuchi, H., Tanai, K., Matsumoto, K., Sato, H., Ueno, K. and Tetsu, T. :”Hydraulic Characteristics of Buffer Material II -The Influence which Saline Water Exert on Hydraulic Properties of Bentontie buffer material -“, JNC TN8430 2003-002 (2004). (in Japanese)

Kikuchi, K. and Tanai, K. : “Basic Characteristic test of Buffer/Backfill material under Horonobe Groundwater Condition”, JNC TN8430 2004-005 (2005). (in Japanese)

Kitagawa, S., Kajiwara, Y., Suzuki, T. and Kawahara, K. : “Success by Double Supporting Method Against Squeezing Ground - Tomikura Work Section in Iiyama Tunnel on Hokuriku Shinkansen Line -”, Tunnels and Underground Vol.31, No.11, pp.7-16 (2003). (in Japanese)

Konishi, K., Nakayama, M., Mihara, M., Yoshida, Y., Iriya, K., Akiyoshi, K. and Noda, M. : “Study on

Applicability of Low Alkaline Shotcrete in Horonobe URL project”, JAEA-Research 2006-040 (2006). (in Japanese)

Kubo, H., Kuroki, Y. and Mihara, M. : “Experimental Investigation on Alteration of Bentonite by Concrete Pore Fluids”, Tuschi To Kiso, Vol.46, No.10, pp.31-34 (1998). (in Japanese)

Kubota, S., Noda, M., Toida, M. and Iwasa, K. : “Feasibility Study on Construction of Horonobe URL”, JNC TJ1400 2002-003 (2003). (in Japanese)

Kurihara, Y., Yui, M., Tanai, K., Kawakami, S., Sugita, Y., Taniguchi, N., Hirai, T., Ogawa, T., Aoyagi, S., Matsui, H., Nakayama, M. and Fujishima, A. : “Design Studies on the Engineered Barrier System and on the In-situ experiments under the Conditions of Geological Environment in Horonobe”, Research document JNC TN8400 2004-006 (2004). (in Japanese)

Kurikami, H., Takeuchi, R. and Seno, S. : “Groundwater Flow Analysis of Horonobe underground Research Laboratory project”, JNC TN5400 2005-003 (2005). (in Japanese)

Martino, J., Chandler, N., Dixon, D., Sugita, Y., Vignal, B. and Hansen, F. : “The Tunnel Sealing Experiment and Application to Repository Seal Design”, IHLRWM 2003, Las Vegas, NV, March 30-April 2, pp.602-609 (2003).

Matsui, H., Tanai, K., Kawakami, S., Sato, T., Nishimura, M., Aoyagi, S., Fujita, T., Taniguchi, N., Kikuchi, H., Matsumoto, K. and Yui, M. : “Estimation of the Applicability on the Disposal Technology under Real Geological Environment”, JNC TN5400 2005-004, (2005a). (in Japanese)

Matsui, H., Yamamoto, T., Sugiyama, K., Yamamoto, T., Mukai, A. And Sakuma, K. : “In-situ Stress Measurements in Deep Neogene Argillaceous Sedimentary Formations”, Proceeding of the 42th Japan National Conference on Geotechnical Engineering, pp.971-972 (2005b). (in Japanese)

Matsui, H. : “Horonobe Underground Research Laboratory Project The Plan for the In-situ Experiments in Phase II and Phase III in/around URL”, JNC TN5400 2005-014 (2005). (in Japanese)

Matsumoto, K. and Tanai, K. : “Extrusion and Erosion of Bentonite Buffer”, JNC TN8400 2003-035 (2004).

Matsumoto, K. and Tanai, K. : “Extrusion and Erosion of Bentonite Buffer (II) - Evaluation of Extrusion process of Bentonite Buffer in Horonobe Groundwater (HDB-6) -”, JNC TN8400 2004-026 (2005). (in Japanese)

Nakayama, G., Nakamura, N., Fukaya, Y., Akashi, M. and Ueda, H. : “Assessment of Crevice Corrosion and Hydrogen-Induced Stress-Corrosion Cracks in Titanium-Carbon Steel Composit Overpack for geological Disposal of High-Level Radioactive Waste”, EFC No.36, Proceedings of International Workshop on “Prediction of Long Term Corrosion Behaviour in Nuclear Waste Systems”, pp.373-394 (2002).

Nakayama, M., Fujishima, A., Iriya, K. and Kurihara, Y. : “Development of Low Alkaline Cement using Pozzolanic Action”, Proceeding of 22th Annual Meeting of Atomic Energy Society of Japan Hokkaido Branch, pp.18-19 (2004). (in Japanese)

Niunoya, S. and Matsui, H. : ”Geochemical Conceptual Model on Neocene Diatomaceous Rock mass Based on Laboratory and In-situ Measurement”, Proceedings of the 35th Symposium on Rock Mechanics, Tokyo, pp. 177-182 (2006).

Nuclear Waste Management Organization of Japan (NUMO): "Proceedings of the International Workshop on Bentonite-Cement Interaction in Repository Environments", Nuclear Waste Management Organization of Japan, 14-16, April, 2004, Tokyo, Japan, NUMO-TR-04-05 (2004a).

Nuclear Waste Management Organization of Japan (NUMO): “Development of Repository Concepts for Volunteer Siting Environments”, NUMO-TR-0403 (2004b). (in Japanese)

Okubo, S. : “Investigation on Evaluation Methods for Long-Term Stability of Near-field Rock Mass”, JNC TJ8400 2003-081 (2004). (in Japanese)

Okubo, S. : “Investigation on Evaluation Methods for Long-term Stability of Near-field Rock Mass”, JNC TJ8400 2004-018 (2005). (in Japanese)

Owada, H., Mihara, M., Kuroki, Y. and Arimoto, K.: “Degradation Studies on Granite in Alkaline Solution”, JNC TN8400 2000-027 (2000). (in Japanese)

Parkhurst, D.L. : "User's Guide to PHREEQC” - A Computer Program for Speciation, Reaction-Path, Advective-Transport, and Inverse Geochemical Calculations, U.S.Geological Survey, Water-Resources Investigation Report 95-4227 (1995).

Pearson, F.J., Arcos, D., Bath, A., Boisson, J-Y., Fernandez, A.M., Gabler, H.E., Gaucher, E., Gautschi, A., Griffault, L., Hernan, P. and Waber, H.N. : "Mont Terri Project - Geochemistry of Water in the Opalinus Clay Formation at the Mont Terri Rock Laboratory", Reports of the FOWG, Geology Series, No.5 (2003).

Power Reactor and Nuclear Fuel Development Corporation (PNC): “Research and Development on Geological Disposal of High-level Radioactive Waste”, First Progress Report H3, PNC TN1410 92-081, (1992). (in Japanese)

Radioactive Waste Management Funding and Research Center (RWMC): “Overall R&D Map for Geological Disposal of HLW”, RWMC-TRJ-04005-1 (2002). (in Japanese)

Sakurai, S. and Adachi, N. : “NATM for Urban Tunnels”, Kajima Press. (1988). (in Japanese)

SKB : “Äspö Hard Rock Laboratory Planning Report for 2003”, Swedish Nuclear Fuel and Wastes Management Co. (SKB), IPR-03-24, pp.37-39 (2003).

Sekiguchi, H. and Ohta, H. : “Induced anisotropy and time dependency in clays”, Proc. 9th ICSMFE , special session9, pp.229-237 (1977).

Sellin, P. : "What are the requirements of the backfill from the point of long-term safety?", Backfill Requirements in KBS-type repository -A POSIVA/SKB Workshop, Swedish Nuclear Fuel and Wastes Management Co. (SKB), IPR-02-05, pp.31-37 (2002).

Shibata, M., Sasamoto, H., Jintoku, T. and Yui, M. : "Status of Assessment Technique for Long-term Stability of Buffer material", JNC TN8400 2004-010 (2004). (in Japanese)

Sugino, H., Fujita, T., Taniguchi, W., Hasegawa, H. and Iwasa, K. : "Design of the HLW Buffer", JNC TN8400 99-096 (1999). (in Japanese)

Sugita, Y., Fujita, T., Tanai, K., Hasegawa, H., Furuichi, M., Okutsu, K. and Miura, K. : "Backfilling of the Underground Facilities on a Disposal Site", JNC TN8400 99-039 (1999).

Sugita, Y., Kurihara, Y., Kawakami, S., Jintoku, T. and Yui, M. : "Studies on the performance Guarantee for the Engineered Barrier System on Geological Disposal of High-level Radioactive Waste (Vol. 2)", JNC TN8400 2003-015, (2003). (in Japanese)

Sugita, Y., Fujita, T., Takahashi, Y., Kawakami, S., Umeki, H., Yui, M., Uragami, M. and Kitayama, K. : "The Japanese Approach to Developing Clay-based Repository Concepts", Clays in Natural & Enin preparation (2005).

Takenaka Corporation : "MuDIAN Theoretical Manual -Version 2.0-", pp.13-74 (1993). (in Japanese)

Taniguchi, N., Kawakami, S. and Morita, M. : "Passivation Condition of Carbon Steel in Bentonite/sand Mixture", JNC TN8400 2001-025 (2002). (in Japanese)

Taniguchi, N., Kawasaki, M., Kawakami, S. and Kubota, M. : "Corrosion Behaviour of Carbon Steel in Contact with Bentonite under Anaerobic Condition", Proc. of EUROCORR 2004, 04-083 (2004).

The Federation of Electric Power Companies of JAPAN and Japan Atomic Energy Agency (JAEA) : "Second Progress Report on Research and Development for TRU Waste in Japan", JAEA-Review 2007-010 (2007).

Yamamoto, T., Shimo, M., Fujiwara, Y., Hattori, H., Tadokoro, T., Iwama, H., Nago, M. and Kumamoto, S. : "HDB-1 Borehole Investigations in Horonobe Underground Research Center", JNC TN1400 2002-010 (2002). (in Japanese)

Yamamoto, T., Shimo, M., Fujiwara, Y., Hattori, H., Nago, M., Tadokoro, T. and Kuji, M. : "Borehole Investigations of HDB-3 in the Horonobe Underground Research Program", JNC TJ5420 2004-003 (2003). (in Japanese)

Yamamoto, T., Shimo, M., Fujiwara, Y., Hattori, H., Nago, M., Tadokoro, T. and Nakagaki, S.: "HDB-6 Borehole Investigation in the Horonobe Underground Research Program", JNC TJ5400 2005-004 (2004a).

(in Japanese)

Yamamoto, T., Matsui, H., Horiuchi, Y. and Timinaga, E. : “Thermal Properties on Soft Sedimentary Rock of Horonobe Underground Research Program”, Proc. of the 34th Symposium on Rock Mechanics, pp.153-158 (2004b). (in Japanese)

Yoshida, Y. and Mihara, M. : “The Study on the pH Behavior of the HFSC Leached Solution - The Development of Model Considering the Pozzolanic Reaction -”, JNC TN8400 2005-013 (2005). (in Japanese)

Yoshino, N., Miyanomae, S., Inoue, H. and Nashimoto, Y. : “A Study about the Long-Term Stability of Sedimentary Rock”, JNC TJ8400 2005-004 (2005). (in Japanese)

Yui, M., Azuma, J. and Shibata, M. : “JNC Thermodynamic Database for Performance Assessment of High-level Radioactive Waste Disposal System”, Japan Nuclear Cycle Development Institute, JNC TN 8400 99-070 (1999).

3. Development of advanced safety assessment methodologies - confirmation of the applicability of safety assessment methods for mass transport analysis -

3.1 Objectives and approach

3.1.1 Objectives

The R&D conducted prior to the H12 Report included evaluation and development of investigation and analytical techniques in terms of their applicability in the area of geological disposal, without considering a specific geological environment; the aim was to demonstrate the generic feasibility of geological disposal in Japan. The methodology of safety assessment in the H12 Report was therefore based on a simplified hypothetical conceptual model of mass transport pathways in the geosphere. The parameters for this model were determined based on generic geological information, meaning that representation of large-scale hydrogeological features consisting of several individual structures, as observed in actual cases, would be difficult. The study was therefore carried out assuming that, without consideration of complex geological structures, the repository would be located at a distance of 100 m from major water conducting faults associated with highly permeable fractured zones, and that nuclides transported by groundwater from the repository through the host rock would migrate upwards in a fault located downstream of the repository and would reach the aquifer taking the shortest pathway. A nuclide migration model was developed based on this assumption and the parameters required for the analysis were determined using information from the existing literature and the investigations conducted in the Kamaishi Mine and the Tono area (JNC, 1999).

As described in section 1.1.1, R&D after the H12 Report has been pursued based on the Generic Program for Research and Development on Geological Disposal of HLW (JNC Assessment Committee for R&D, 2001). For one of the goals of the program - confirmation of the applicability of geological disposal technologies in relevant geological environments - the program specifies that the reliability of these technologies should be demonstrated by applying the technologies developed up to the time of the H12 Report to relevant geological environment conditions. As part of this program, a hydrogeological model was developed using data for a specific geological environment; based on this, a groundwater flow analysis was conducted to develop a mass transport analysis model and to determine the parameters required for the analyses. It was expected that, through this study, the level of understanding of key features of the geological environment would improve with the progress of stepwise investigations, including excavation of shafts and tunnels. Also expected was the evaluation of a series of methods for investigations and analyses based on an evaluation of the uncertainties associated with the data and models. Table 3.1.1-1 shows a comparison between the H12 Report and this study.

Deriving models and parameters through interactions among research teams from different R&D disciplines is essential for achieving the goals described above. The focus was therefore placed on collaboration among these teams. As part of this, a mass transport analysis was carried out based on the results obtained in the Horonobe URL project. Although safety assessment includes scenario analysis, deterministic and probabilistic safety assessments etc, the main focus in this report was on the mass transport analysis. The objectives of the mass transport analysis were:

(a) To demonstrate a mass transport analysis approach based on the results of surface-based investigations and to identify procedures for developing analysis models and input parameters.

(b) To highlight important issues relating to investigation of the geological environment and development of models of the geological environment to be reflected in planning the program for the next phase through studies of sensitive factors or factors with large uncertainty in the mass transport analysis.

In other words, the analysis was conducted with the aim of evaluating the applicability of the methodology presented in the H12 Report to relevant geological environments and contributing the knowledge obtained to implementation of disposal project and to safety regulations.

The mass transport analyses in this study require modeling of hydrogeological structures (including the distribution of permeability) that would affect groundwater flow, based on data from investigations in boreholes, estimation of the groundwater flow-field by groundwater flow analysis and development of a model that can reproduce mass transport behavior in the geological environment of interest. Data are also required on geological environment conditions, including hydrochemistry and physico-chemical rock properties, relevant to mass transport based on information from investigations of actual geological environment and groundwater flow analysis, for setting parameter values to be used in the mass transport analysis for such as groundwater velocity, transport distance, distribution coefficient, solubility and diffusion coefficient.

Table 3.1.1-1 Comparison of this study with H12 Report

	H12 Report	This Study
Aim	Demonstrating the technical reliability of geological disposal in Japan	Confirmation of the applicability of safety assessment methodology
Data	Hypothetical data of geological environment based on literature survey mainly	Actual data of geological environment obtained from the phase I investigation of Horonobe URL project
Methodology	Setting the analytical models and parameters deterministically (Applied 1-D parallel plates model to mass transport analysis assumed fractured rock in the reference case)	Basically applied the methodology of H12 report (Applied 1-D porous media model to mass transport analysis because the objective area consists of sedimentary rock)

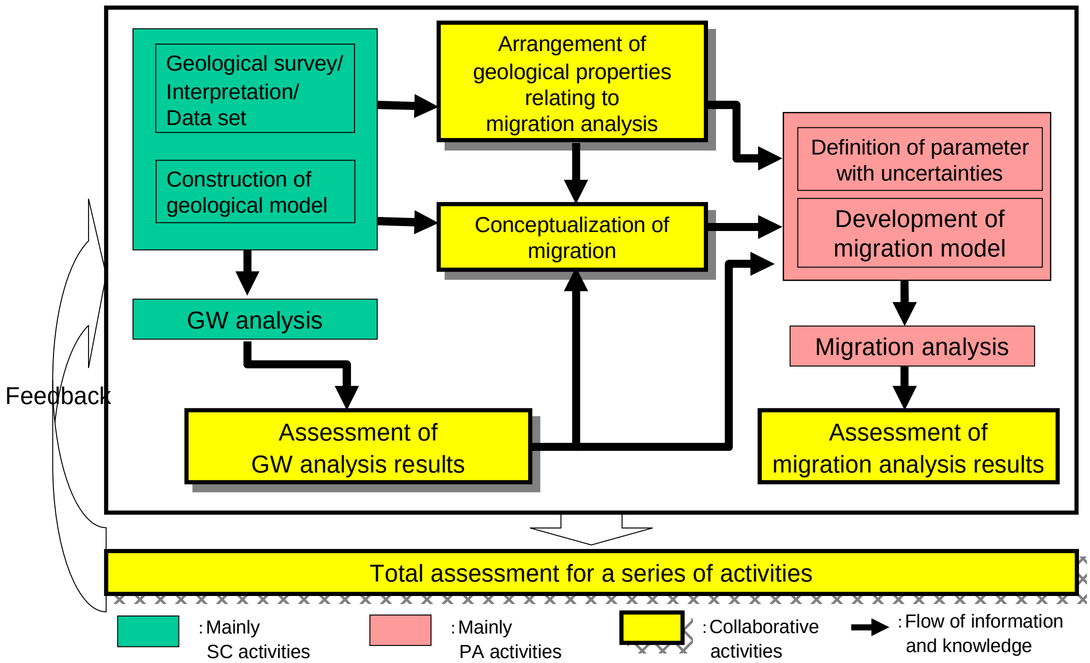


Figure 3.1.1-1 A framework and a preliminary work flow

The study was started with formulation of a framework and a preliminary work flow (Figure 3.1.1-1) for a series of tasks to be performed for mass transport analysis using geological information obtained in the Horonobe URL project. Based on geological information available up to the end of March 2005, details of each work unit in the work flow and the information to be transferred between the work units were then clarified. In addition, a cooperation program among the research teams, sucha as a teams responsible for geological investigations and for mass transport analysis, was formulated. In adition, the important issues and aspects for the cooperation program were listed.

The modeling and analyses of mass transport used same domains as specified in the H12 Report, except for the biosphere, which consist of the main flow paths including the engineered barriers, the surrounding rock and faults. In order to make a study of the relevant geological environment, the depth where mass transport would start needs to be specified. In this study, the depth was set to approximately 450 m, at which the rock in the Horonobe project proved to have sufficient thickness and mechanical stability (refer to section 2.2.1). When conducting a safety assessment based on relevant geological environment conditions, information for the assessment should be based on the design implemented according to the geological environment conditions. However, in this preliminary study on the application of safety assessment methodology only the depth was taken into account as a starting point of the mass transport analysis. Also the following assumptions were made:

- Steady state conditions
- no consideration of the evolution of flow and geochemistry over time;
- No consideration of fracture model for transport analysis;
- No consideration of perturbations due to colloids, gas, etc.;
- No consideration of disruptive events such as earthquakes;
- Independent, 1-D transport paths through the geosphere with no consideration of spreading or mixing between transport paths in different geospheres (transverse dispersion).

3.1.2 Study approaches

The Horonobe URL project (JNC, 1998) included investigations for selecting the URL area (main research areas, each measuring around 2 or 3 square km) (Yamasaki et al., 2002) and investigations in and around the selected site (section 1.2.2). The mass transport analyses in this study mainly used the results of the latter, i.e. field investigations (geological investigations, airborne and ground geophysical surveys and borehole investigations), various laboratory tests (physical tests, mineral analyses, chemical analyses and mass transport tests) and groundwater flow analyses. The data obtained from these investigations included geological structures, hydrological properties, groundwater chemistry, rock mechanics and mass transport properties.

To achieve the objective (a) described in section 3.1.1, the focus was on improving the knowledge base developed on a trial-and-error basis covering all areas from investigations of the geological environment to mass transport analyses. For the objective (b), the points to be addressed in Phase I, as well as those to be reflected in the Phase II program (investigations during excavation of the underground facility) were identified through a review of factors that would have a significant effect on the mass transport analysis results and on the associated range of uncertainty.

It is important for all the information on the geological environment, including quality of the data, the concept for data interpretation, a description of heterogeneity and working hypotheses for the development of geological models, to be integrated appropriately in the mass transport models and analyses. It is also important to evaluate the analysis results against the background of the current level of knowledge on geological characteristics and identified uncertainties at the time. In every stage from investigations and analysis of geological environment through mass transport analysis, results should be reflected appropriately in the next steps. By iterating a series of procedures in the course of the study, the evolution of uncertainties associated with the work or analysis results should become clearer. This allows approaches for reducing uncertainties and challenges in terms of optimizing of analytical methods to be clarified.

This study followed the work flow shown in Figure 3.1.1-1, together with investigation of the geological environment according to the integrated data flow shown in Figure 3.1.2-1, focusing on the following four items:

(a) Clarification of details of the individual work units and the necessary information transfer among them
(b) Development of a mass transport model
(c) Setting the parameter values to be used in the mass transport analyses
(d) Evaluation of the results of the analysis based on actual geological environment conditions

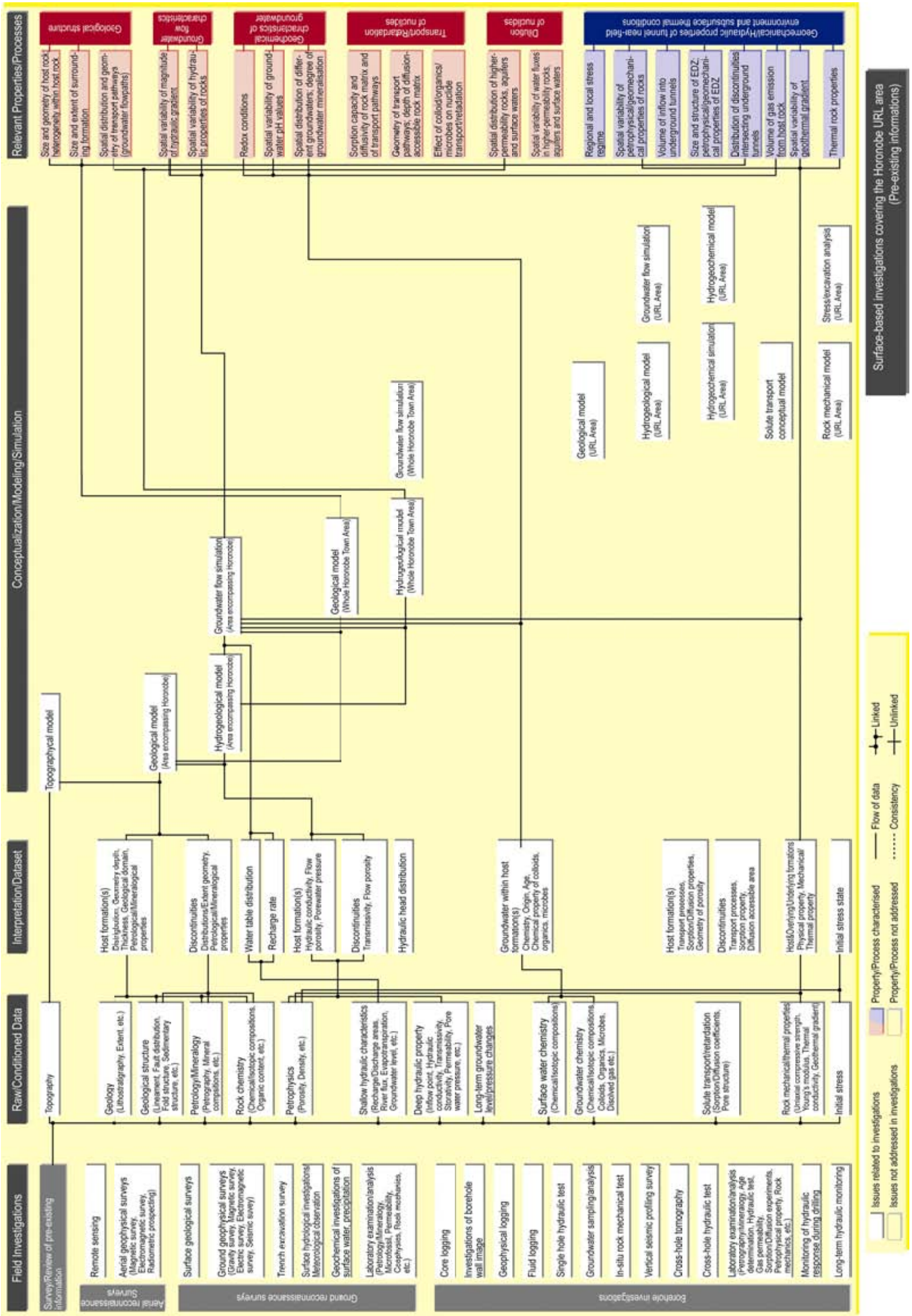


Figure 3.1.2-1 An Example of geosynthesis data flow diagram

3.2 Information on the geological environment

Of the work units shown in Figure 3.1.1-1, this section outlines the investigation stages, the tests and investigations conducted in each stage and the resulting information.

3.2.1 Outline of surface-based investingation

Investigations in Phase I of the Horonobe URL project were divided into two stages: one for selecting the URL area (main research areas, each about 2 to 3 square km) and one in and around the selected area. Section 3.2.2 summarizes the information obtained from the latter, which involved (a) surveys of existing information, (b) surface-based investigations and (c) borehole investigations, as well as modeling and analyses of the geological environment based on the results of investigations.

3.2.2 Information from investigations in the URL area (refer to Volume 1, Section 4.2)

(1) Surveys of existing information

Information on the geological environment in and around the URL area, including geological structure, groundwater flow characteristics, hydrochemistry and rock mechanical properties, was compiled based on data from literature compiled for selection of the URL area. The geological map was also updated accordingly (Figure 3.2.2-1).

In order to develop a more specific investigation program, models of geological and hydrogeological structures were developed for an area covering around square 30 km from the east end of the Teshio River and the west end of the Teshio Mountains (a coastal area), covering the entire Horonobe Town area, and scoping analyses were conducted of groundwater flow using these models (Imai et al., 2001). Figure 3.2.2-2 shows the distribution of hydraulic heads and groundwater flowpaths in and around the URL area, based on the analysis results. It indicates that the hydraulic heads in the eastern area are higher than those in the west and that the main groundwater flow direction is from east to west. It also suggests that, due to local relief, the distribution of all hydraulic heads is fairly complex and the flow direction varies depending on depth and location in and around the URL area. The streamlines obtained from the analysis imply that groundwater flow in this area is governed by the local flow system at shallower depth and by the regional flow system at greater depth (Kurikami et al., 2005). A conceptual diagram of the groundwater flow system in and around the URL area based on this information is shown in Figure 3.2.2-3.

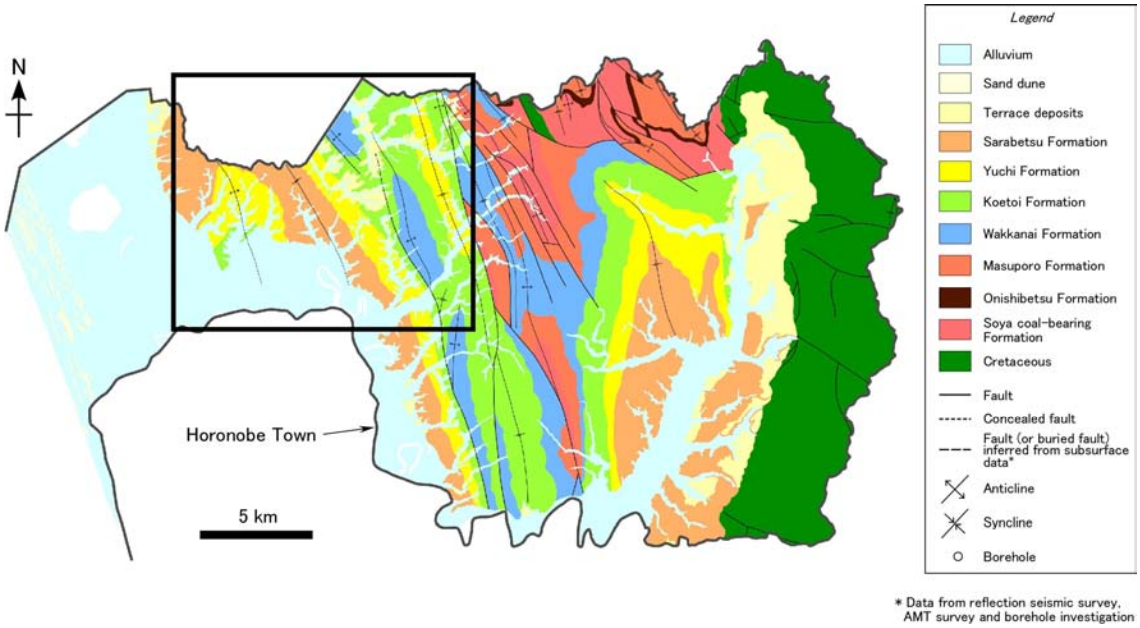


Figure 3.2.2-1 Geological Map around the Horonobe Area
Square area: More detail geological map with deep borehole location
is shown in Figure 3.2.2-5

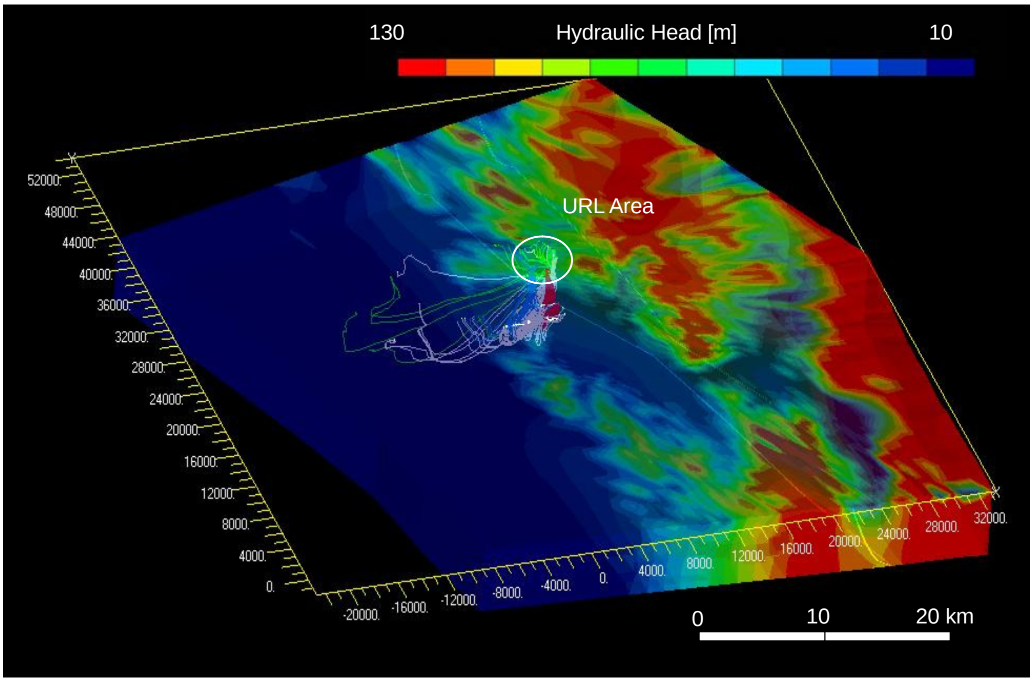


Figure 3.2.2-2 Hydraulic Head Distribution and Groundwater Flow Path from URL Area
(Numerical Results)

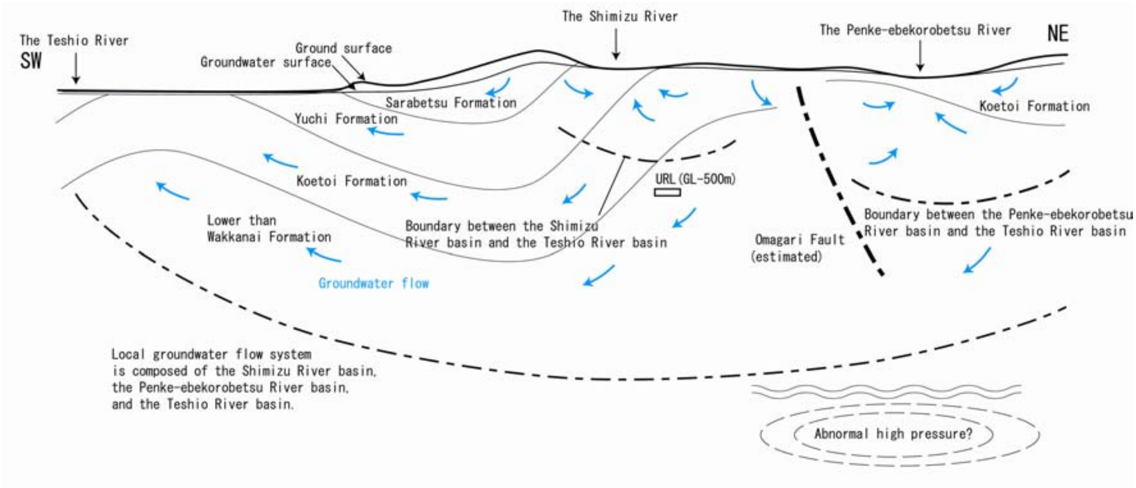


Figure 3.2.2-3 Schematic illustration of Groundwater Flow around URL Area
(about 10km scale area)

(2) Surface-based investigations

Based on the preliminary information on the geological environment compiled from surveys of existing data, topographic investigations and surface and geophysical explorations were conducted to acquire information mainly on the near-surface geological environment in terms of geological heterogeneity and features that are critical for determining the mass transport pathways. These included electromagnetic and reflection seismic surveys. The amount of groundwater recharge was estimated in order to determine the upper boundary condition for the groundwater flow analysis, and the hydrochemistry of surface waters was analyzed for use in developing hydrochemical models.

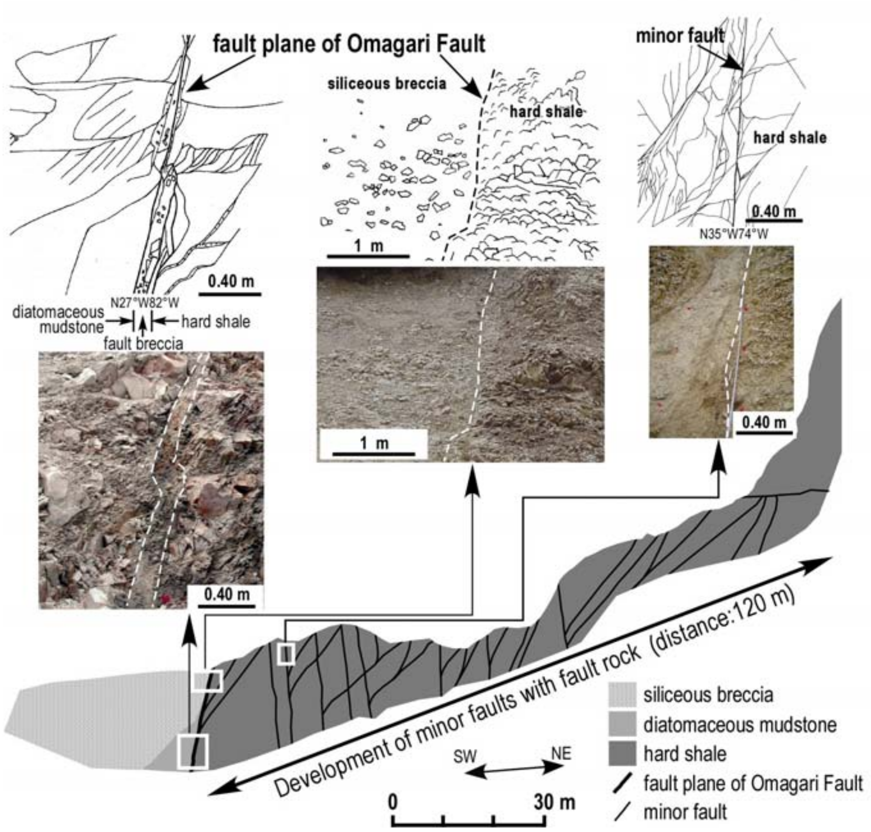


Figure 3.2.2-4 Sketches at the fault outcrop of the Omagari Fault(Ishii and Yasue, 2005)

The Omagari Fault, a large discontinuous geological feature located in the URL area, could function as a preferential pathway for mass transport. Surface and gas explorations were therefore conducted to determine its location and characteristics.

Surface explorations had difficulty in identifying the Omagari Fault due to poor outcropping in and around the URL area, but it was finally identified in a quarry located 5 km to the south of the URL area (Figure 3.2.2-4). Reflection seismic and AMT electromagnetic surveys conducted at this stage could not determine the location and geometry of the Fault.

(3) Borehole investigations

The borehole investigations were focused on verifying the spatial distribution of discontinuities and geological structures predicted based on the investigations to date and more detailed surveys of the three-dimensional distribution of permeability and groundwater chemistry and rock mechanical properties. Based on the results, models were developed for the geological and hydrogeological structures, hydrochemistry and rock mechanics around the URL area.

Figure 3.2.2-5 shows the location of the boreholes drilled in and around the URL area. Information on lithofacies obtained from the borehole investigations was basically consistent with the results of the surface-based investigations. Of the discontinuities that would be important for evaluating flowpaths, the Omagari Fault was investigated in terms of its three-dimensional distribution in and around the URL area as shown in Figure 3.2.2-6. This figure is based on a comprehensive interpretation of investigation results to date, i.e. the results of reflection seismic and AMT electromagnetic surveys conducted in the previous phase, the depth profile of lithofacis boundaries and information on factors causing variable rock resistivity obtained from borehole investigations, the results of observation of fractures in the boreholes and descriptions of the faults in outcrops.

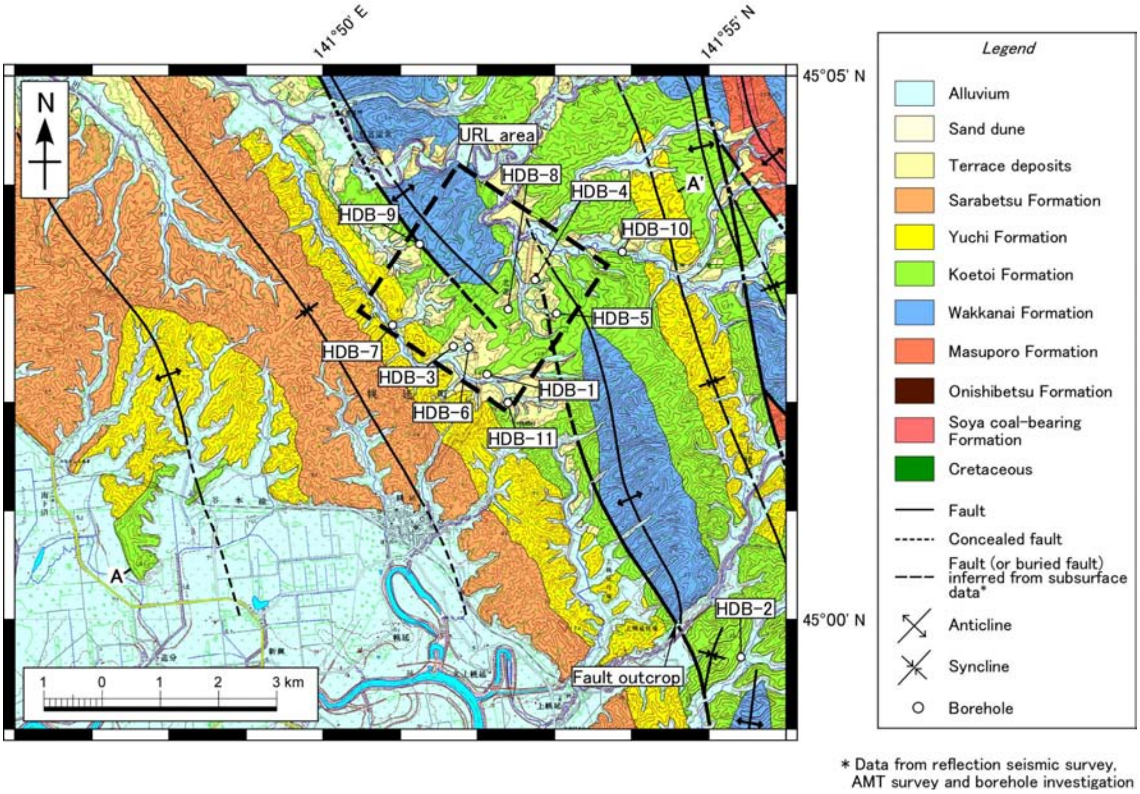


Figure 3.2.2-5 Location of Deep Boreholes for the Investigations

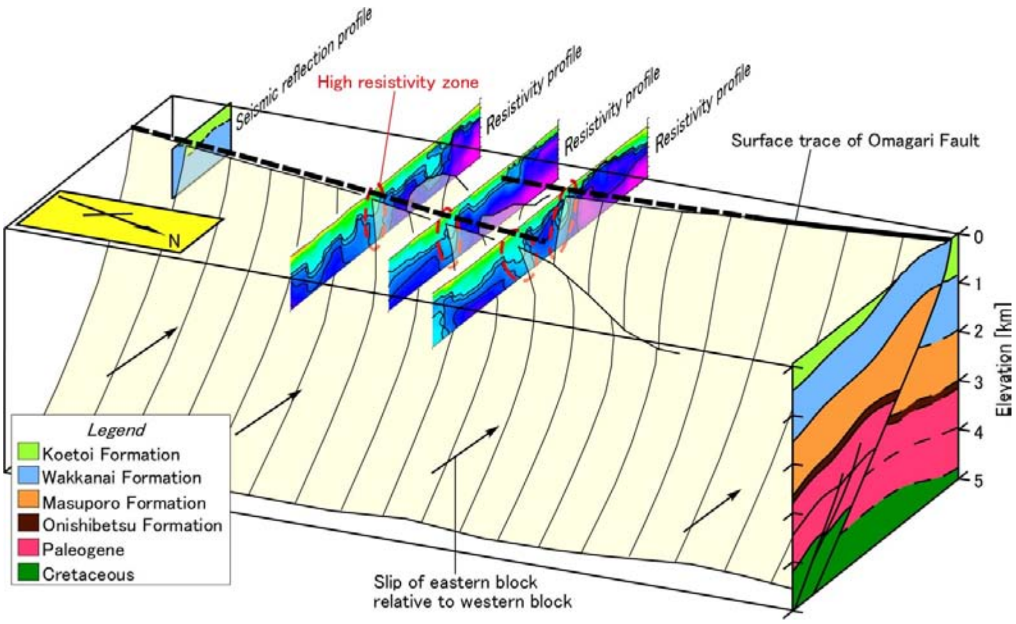


Figure 3.2.2-6 Schematic diagram of the Omagari Fault and around URL area (Ishii and Yasue, 2005)

High resistivity zones found in the profile in the figure are assumed to be zones where freshwater is distributed down to depth, based on the relationship between resistivity and chloride ion concentration in the porewater. The three-dimensional distribution of the Omagari Fault was estimated based on the distribution of these high resistivity zones. The Omagari Fault in and around the URL area is assumed to be overstepped at the ground surface and converged underground (Ishii and Yasue, 2005). The estimated location of the Fault was almost identical with locations where seepage of carbon dioxide was observed during the gas explorations conducted in the previous stage (2) above.

In terms of small-scale discontinuities, the distribution of minor strike-slip faults intersecting the bedding plane of the Omagari Fault at a high angle and bedding faults have been observed. The former tend to be densely distributed. For minor faults, more than three faults every 10 m were identified by EMI logging, each with dominant strike slips intersecting bedding plane at a high angle. These were defined as fracture zones in this stage of the study. The results of in-situ hydraulic tests suggest that the hydraulic conductivities of the fractured zones in the Wakkanai Formation are one to four orders of magnitude higher than those of the surrounding rock (JNC, 2004). Figure 3.2.2-7 shows the depth distribution of hydraulic conductivity in the Wakkanai Formation for test intervals with fractured zones (○) and without (●) respectively. This figure indicates that the variability and depth dependence of the data observed in areas with fractured zones are important factors constituting the hydrological characteristics of the entire Wakkanai

Formation. Fractured zones have also been observed in the Koetoi Formation, but there is no evidence to date that they have a significant influence on the hydraulic conductivity in this formation. This may be because, compared to the Wakkanai Formation, fractures in the Koetoi Formation tend to be closed and have not developed significantly due to the soft nature of the rock (Shimo et al., 2004).

The geological structure model developed for this area is shown in Figure 3.2.2-8. The model does not include fracture zones due to lack of information on their distribution and influence on hydraulic conductivity. The indicated Omagari Fault is based on a three-dimensional distribution that includes the results of surface explorations, geophysical surveys and borehole investigations. The results of gas explorations and borehole investigations support the view that the Omagari Fault and minor faults (especially fracture zones) intersecting the Fault at a high angle form major groundwater flowpaths. In order to clarify their hydrological and mass transport properties in more detail, further studies including in-situ tests are required.

The hydrogeological model was developed based on the geological structure model as well as interpretation of hydrological data (Figure 3.2.2-9) and a groundwater flow analysis was conducted on this basis (Kurikami et al., 2005).

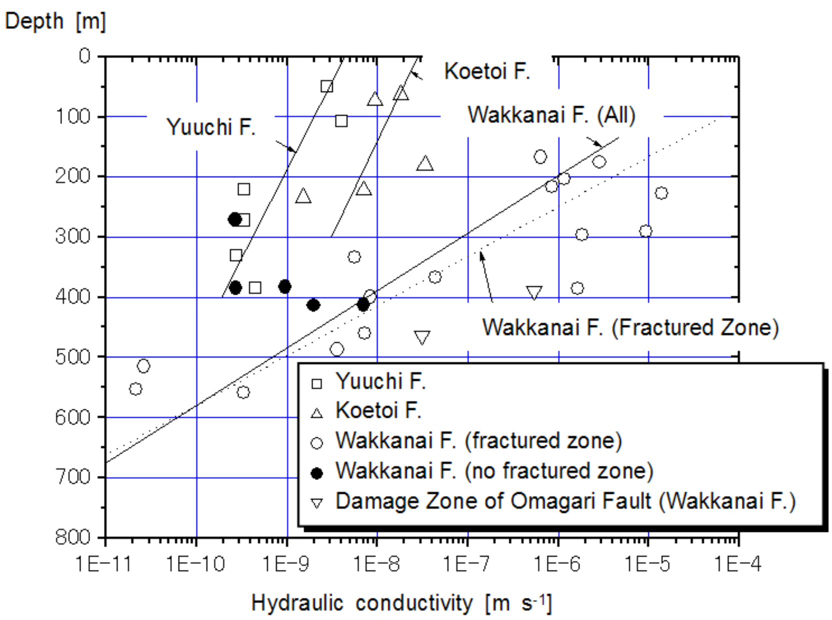


Figure 3.2.2-7 Depth dependency of hydraulic conductivity (Kurikami et al., 2005)

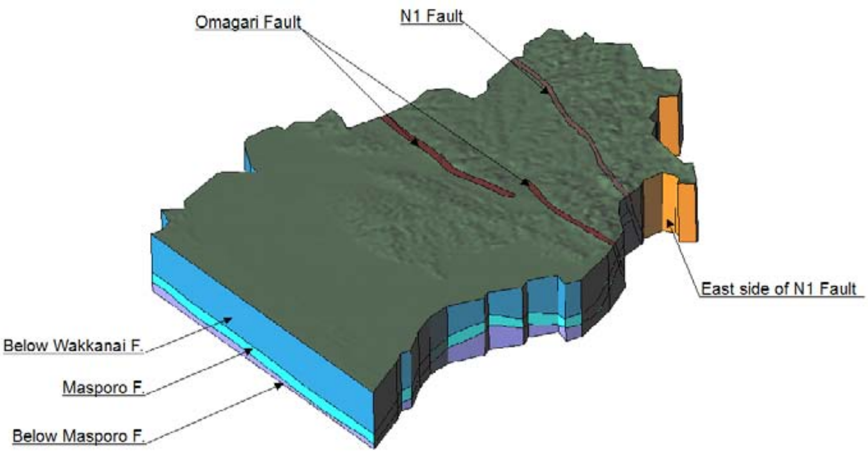


Figure 3.2.2-8 Geological Structure Model (Shimo et al., 2005)

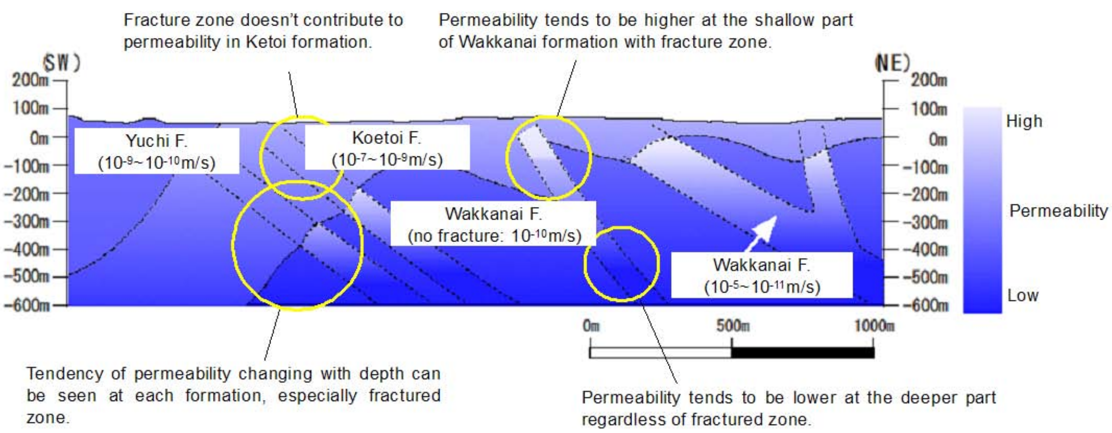


Figure 3.2.2-9 Conceptual Hydrogeological Model (Takeuchi et al., 2004)
Zones surrounded by dotted lines are estimated as fractured zones.

The regional-scale area measuring approximately 30 km scale of square area, including Horonobe Town, defined based on the study results so far was selected as the domain for the analyses (Figure 3.2.2-10). Hydrogeological zones analyzed include the lower part of the Masuporo Formation, the Masuporo, Wakkanai, Koetoi, Yuchi and Sarabetsu Formations, the near-surface zone and the Omagari Fault. Considering uncertainties in the measured data and the lack of data on hydraulic conductivity for some formations and upper boundary conditions, the analysis was conducted for cases (Case 001 to 018) with different hydraulic conductivities in each formation and boundary conditions. Table 3.2.2-1 lists the cases and Table 3.2.2-2 shows the conditions specified for each case and the reasons for this. Case 001 is the reference case. The conditions for the reference case were determined as follows, based on results of borehole investigations.

(a) The hydraulic conductivity of the Omagari Fault measured in borehole HDB-4 was that for the damaged zone of the fault and the values could be directly used.

(b) For the Yuchi Formation, the depth dependence of hydraulic conductivity was taken into account, because the deviation between actual data and approximation line considering the correlation with depth was smaller than logarithmic average of hydraulic conductivity.

(c) For the Koetoi Formation, the logarithmic average was used because the depth dependency was not so significant compared to that of the Yuchi Formation.

(d) For the Wakkanai Formation, hydraulic conductivity was determined for the whole formation without differentiating between parts with and without fracture zones. Depth dependence should be taken into account.

(e) The upper boundary condition was defined as ground surface level.

(f) The lower boundary condition was defined as an impermeable boundary 5,000 m below sea-level. The lateral boundary condition toward the sea was defined as a permeable. The lateral boundary for the other sides was defined as an impermeable boundary because most of the areas were bordered by mountain ridges or valleys.

In Cases 006 to 014, hydraulic conductivities for the Yuchi, Koetoi and Wakkanai Formations were varied within a realistic range from the measured values. Cases 002 to 005 and 015 to 017 are for sensitivity analyses of expected hydraulic conductivities of the formations where no measured data are available. Case 018 is a hypothetical extreme case assuming abnormally high pressure deep underground.

Compared with the hydraulic pressures obtained from hydraulic tests and long-term monitoring, the analysis result for Case 003 proved to be closest to the actual distribution of the hydraulic pressure. In this case, the hydraulic conductivity for the near-surface zone, for which measured data are lacking, was set one order of magnitude lower than that for Case 001. A comparison was also made for recharge rate with data obtained from the surface-based investigations (Seno et al., 2005) and showed a relatively good agreement between the analytical results and the measured data. A correlation was assumed in the Wakkanai Formation between the existence of fracture zones and permeability. Therefore, an analysis taking into account the influence of fracture zones on the anisotropy of the hydraulic conductivity was also conducted as a first-order approximation. Although not used here, for mass transport modeling in fractured media, information such as fracture filling, channeling from observations of core specimens would also be required.

Based on the analysis results for the eighteen cases, streamlines were developed for use in the mass transport analysis. The starting-point of streamlines was assumed to be at the center of three boreholes 390 m below sea-level (approximately 450 m depth). The result is shown in Figure 3.2.2-11. The results for groundwater flow velocity along the flow paths are described in section 3.3.3 (2).

Figure 3.2.2-12 shows the distribution of groundwater chemistry obtained from deep borehole investigations (Yamamoto et al., 2002, 2004a-c, 2005a-c). It indicates that the concentration of dissolved constituents is low in the shallower zone (Na-HCO_3 type) and the salinity is relatively high in the deeper zone (Na-Cl type). The distribution of the chloride ion concentration was estimated using a geostatistical method. The result indicates that freshwater permeates relatively deep underground in the eastern area of the URL site (Figure 3.2.2-13).

Table 3.2.2-1 Cases of Groundwater Flow Analysis

	Alluvium Terrace deposit	Sarabetsu Formation	Yuchi Formation	Koetoi Formation	Wakkanai Formation	Masuporo Formation	Omagari Fault	Boundary condition
Case-001	Sur1	S1	Y1	K1	W1	M1	D1	B1
Case-002	Sur1	S1	Y1	K1	W1	M1	D1	B2
Case-003	Sur2	S1	Y1	K1	W1	M1	D1	B1
Case-004	Sur3	S1	Y1	K1	W1	M1	D1	B1
Case-005	Sur1	S2	Y1	K1	W1	M1	D1	B1
Case-006	Sur1	S1	Y2	K1	W1	M1	D1	B1
Case-007	Sur1	S1	Y3	K1	W1	M1	D1	B1
Case-008	Sur1	S1	Y4	K1	W1	M1	D1	B1
Case-009	Sur1	S1	Y1	K2	W1	M1	D1	B1
Case-010	Sur1	S1	Y1	K3	W1	M1	D1	B1
Case-011	Sur1	S1	Y1	K4	W1	M1	D1	B1
Case-012	Sur1	S1	Y1	K1	W2	M1	D1	B1
Case-013	Sur1	S1	Y1	K1	W3	M1	D1	B1
Case-014	Sur1	S1	Y1	K1	W4	M1	D1	B1
Case-015	Sur1	S1	Y1	K1	W1	M2	D1	B1
Case-016	Sur1	S1	Y1	K1	W1	M1	D2	B1
Case-017	Sur1	S1	Y1	K1	W1	M1	D3	B1
Case-018	Sur1	S1	Y1	K1	W1	M1	D1	B3

Table 3.2.2-2 Evidence for Parameter Setting for Groundwater Flow Analysis

	Note	Setting condition [m s-1]	Specification
Omagari Fault	D1	1.30×10^{-7}	Logarithmic mean of two data in HDB-4
	D2	log(k)=-0.01z-2.98	Assumed as fractured zone in Wakkanai F.
	D3	ky=kz=1.30×10^{-7} kx=1.30×10^{-9}	Assumed orthogonal anisotropy
Yuchi Formation	Y1	log(k)=-0.003z-8.37	Assumed as depth dependent
	Y2	log(k)=-0.003z-9.37	Case of 1/10 of Y1
	Y3	log(k)=-0.003z-7.37	Case of 10 times of Y1
	Y4	7.31×10^{-10}	Logarithmic mean of whole measured data of Yuchi F.
Koetoi Formation	K1	9.07×10^{-9}	Logarithmic mean of whole measured data of Koetoi F.
	K2	9.07×10^{-10}	Case of 1/10 of K1
	K3	9.07×10^{-8}	Case of 10 times of K1
	K4	log(k)=-0.003z-7.55	Assumed as depth dependent
Wakkanai Formation	W1	log(k)=-0.01z-3.91	Assumed as depth dependent
	W2	log(k)=-0.01z-4.91	Case of 1/10 of W1
	W3	log(k)=-0.01z-2.91	Case of 10 times of W1
	W4	1.14×10^{-8}	Logarithmic mean of whole measured data of Wakkanai F.
Masuporo Formation	M1	5.00×10^{-10}	Estimated as high permeability due to gravel and sandstone
	M2	5.00×10^{-8}	Case of 100 times of M1
Sarabetsu Formation	S1	1.00×10^{-6}	Based on pumping test in D-1 (Oshima et al., 1995)
	S2	1.00×10^{-5}	Case of 10 times of S1
Alluvium, Terrace deposit	Sur1	1.00×10^{-6}	Esimated value
	Sur2	1.00×10^{-7}	Case of 1/10 of Sur1
	Sur3	1.00×10^{-5}	Case of 10 times of Sur1
Boundary condition	B1	Side&Bottom: impermeable Seashore: constant head (sea level) Upper seepage area: 1 [mm day^{-1}]	Based on the literature survey
	B2	Upper seepage area: 3 [mm day^{-1}]	Case of 3 times of recharge rate in B1
	B3	Bottom: constant head (+500m)	Assumed residue of excess hydrostatic pressure

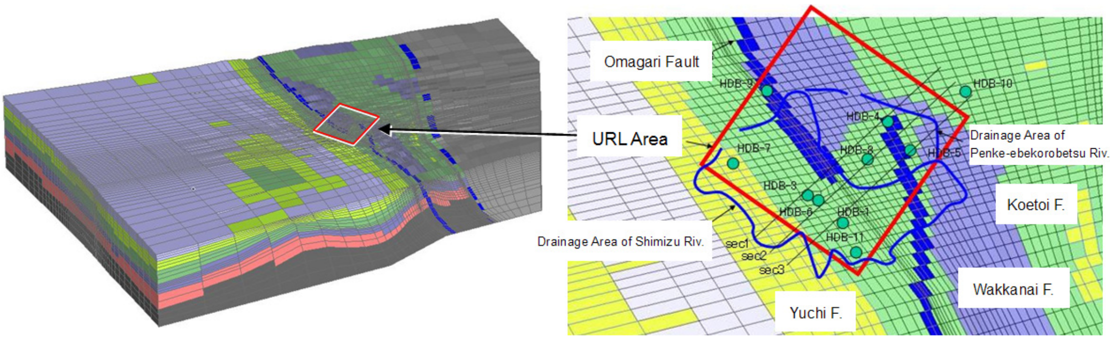


Figure 3.2.2-10 Target Area for Groundwater Flow Analysis

Red square area is about 2km x 2km.

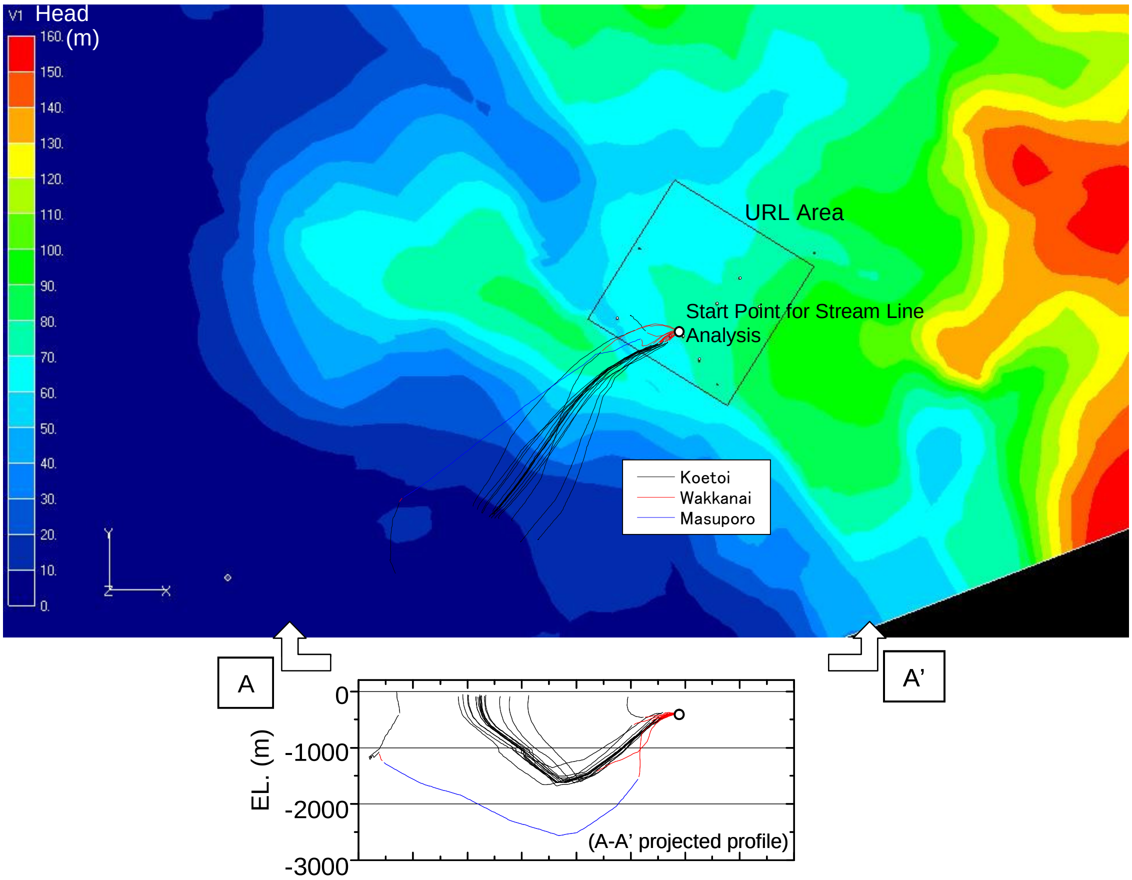


Figure 3.2.2-11 Result of Groundwater Flow Analysis

Upper: Head distribution at EL.-400m (case003) with the streamlines for each case

Lower: Streamlines at A-A' cross section

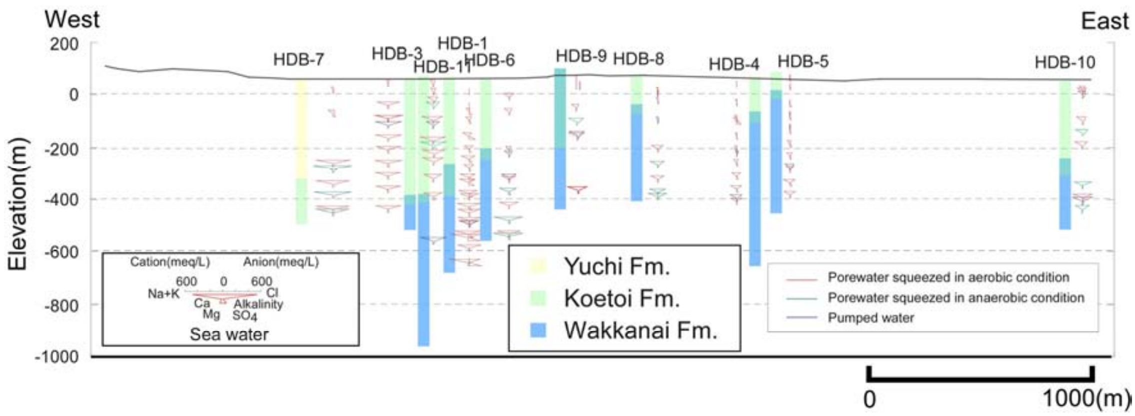


Figure 3.2.2-12 Distribution of Groundwater Chemistry

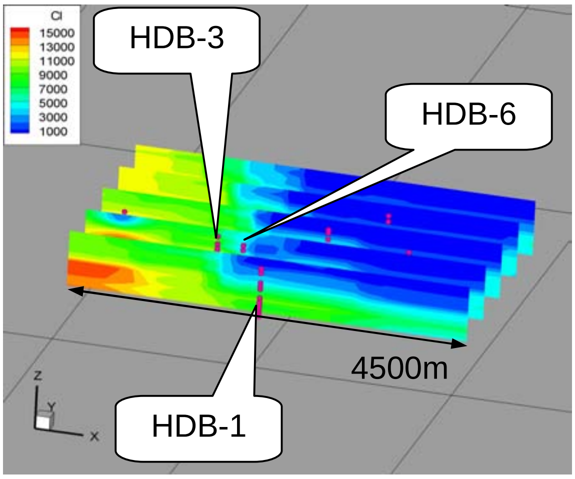


Figure 3.2.2-13 Spatial Distribution of Chloride Concentration Around URL area
HDB-1, HDB-3, HDB-6 are deep drilled boreholes (NEEDS TO ADD LENGTH SCALE!)

3.3 Conceptualization and parameters for the mass transport analysis based on geological information

Of the work units shown in Figures 3.1.1-1 and 3.1.2-1, this section summarizes those involving mass transport analyses based on geological information obtained from the investigations described in section 3.2, i.e. evaluation of the results of groundwater flow analyses, collection of data on geological properties related to mass transport, conceptualization of mass transport, development of mass transport models and determination of parameters for mass transport analyses. Individual work procedures were developed referring to the geological information actually available and the studies carried out with this information are described in the following sections.

3.3.1 Procedure for selecting data for the mass transport analysis

The geological information obtained from the borehole investigations described in section 3.2 was used for the mass transport analysis. Information that can be used directly in the analyses can be classified as follows:

- Information on hydrogeological structures and hydrological properties (from the dataset on hydrological properties and the hydrogeological structure model)
    - Concept for the hydrogeological structure model, classification of hydrogeological

structures

- Porosity, hydraulic conductivity

- Information on flowpaths (from the results of the groundwater flow analysis)
  - Transport time and distance
  - Transport route
- Information on groundwater chemistry (from the dataset on hydrochemistry of groundwater and the hydrochemical model)
- Information on mineralogical composition (from the dataset on geological structure)
- Information on rock density (from the dataset on geological structure)
- Information on rock porosity (from the dataset on geological structure)

Based on the work flow shown in Figures 3.1.1-1 and 3.1.2-1 as well as the information described above, a procedure for determining mass transport concepts, models and parameters as described in sections 3.3.2 and 3.3.3 was developed. Firstly, correlations between the items to be determined in individual work units (e.g. concepts, models and parameters) and the information to be used for these items (e.g. investigation and test data and results of groundwater flow analysis) were defined for each work step. Strategies and methods were then defined for each step. Based on this, the necessary models and parameters were determined. How uncertainty was handled in each step is also described in the following sections.

3.3.2 Conceptualization for mass transport

This section describes how the domain for the mass transport analyses is determined (model domain) and how the mass transport model is developed (modeling strategy). Development of models for the mass transport analyses will also be demonstrated based on the domains and strategy in this section.

(1) Model domain

When determining the model domain, the following aspects were considered:

- In order to identify major features comprising the geological structure in the target area that may influence groundwater flow, and to acquire data on their properties, the influence of these features on the mass transport analysis were determined.
- The target area should allow mass transport analyses that are consistent with the H12 Report in terms of the near-field approach, focusing on the performance of the engineered and natural barriers.

As a result, the domain for the mass transport analyses was selected as the rock surrounding the experiment drift planned for the Horonobe URL (in the Wakkanai and Koetoi Formations) in an area measuring approximately 30 km x 15 km. The starting-point of mass transport was assumed to be a depth of 450 m (see section 2.2.1).

Note that the analyses are focused on mass transport within the natural barrier sytem in the area defined above and mass transport in the biosphere was not considered in this study.

Evaluation of the analysis results focused on the release rates from the Koetoi Formation, through the Wakkanai Formation and the Koetoi Formation.

(2) Modeling concept and strategy

The mass transport model was based on a porous medium. As described in section 3.2.2 (3), the fracture zones in the Wakkanai Formation may influence groundwater flow and the groundwater flow analysis based on a porous medium was conducted taking this influence into account. However, information on the distribution of the fracture zones and their influence on permeability was insufficient and they were therefore excluded from this analysis.

The mass transport model included heterogeneities along the trajectory line based on the information on streamlines obtained from the groundwater flow analysis (e.g. Figure 3.2.2-11), as discussed in section 3.2.2(3). The trajectory line here means the flow path expressed as a set of trajectories with time. A segment of the trajectory line in each calculation step is defined as a trajectory interval. The transport distance along the trajectory line for each calculation step is defined as the length of the trajectory interval. The groundwater velocity is calculated from the length of a trajectory interval divided by the time of the calculation step.

(3) Determining the details of the mass transport model

A base scenario where nuclides migration occurs via groundwater flow was examined and all safety functions from the H12 report were assumed.

The mass transport model for the natural barrier was developed for sedimentary rock where no influence of discontinuities such as fracture zones on groundwater flow was assumed. Since information on flowpaths was obtained from the groundwater flow analysis for a porous medium, mass transport was expressed for each trajectory interval using a one-dimensional porous medium model (Figure 3.3.2-1). In this model, it was assumed that the faults are a porous media as used in the H12 Report in one of the perturbation scenarios for the natural barrier.

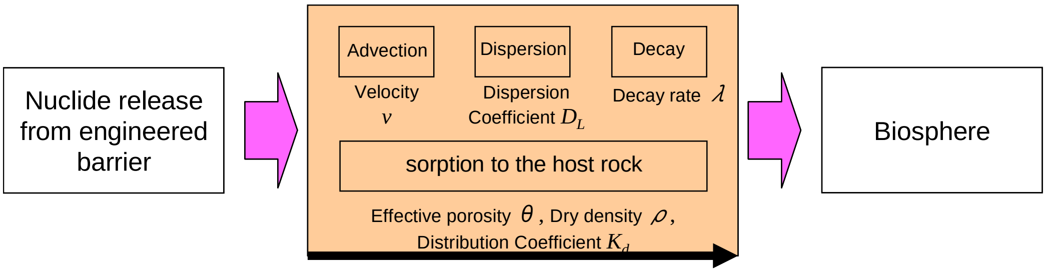


Figure 3.3.2-1 Conceptual Illustration of Mass Transport in Natural Barrier (1-Dimentional Porous Media)

Considering the heterogeneity of the information on the flowpath (trajectory line), the groundwater flow velocity and the transport distance to be obtained from the groundwater flow analysis as described in section (2) above, two options could be considered for the modeling approach to be incorporated in the mass transport analysis as follows:

Approach (a): The heterogeneity of the flowpath information was incorporated directly in the model.

Individual trajectory interval was expressed as a one-dimensional porous medium model and the models were connected in series. The flowpath information for each trajectory interval was then used to conduct the mass transport analysis.

Approach (b): The heterogeneity of the flowpath information was simplified, e.g. by averaging, and then incorporated in the mass transport analysis.

Each geological feature which the trajectory line passes through (Wakkanai Formation and Koetoi Formation) was represented as a one-dimensional porous medium model and the models were connected in series. The flowpath information for each trajectory intervals was then used to conduct the mass transport analysis.

A comparison between these two approaches is shown in Figure 3.3.2-2. Approach (a) allows an objective evaluation of the results and is therefore useful for understanding general trends.

When numerous data are to be handled, however, this may cause complications in the procedure. Approach (b) would be easier to process multiple analytical results with similar trends although a subjective decision would be required for the grouping of the results.

A one dimensional cylindrical radial coordinate system used in the H12 Report (Figures 3.3.2-3 and 3.3.2-4) was used to mass transport models for the engineered barriers coupleing with interface zone to natural barrier.

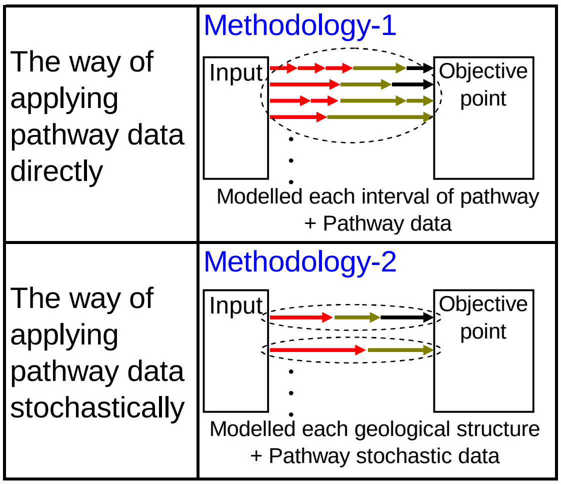


Figure 3.3.2-2 Methodologies of Defining the migration path

Modeling of mass transport for each trajectory interval using one-dimensional porous medium models and modeling of coupling along the trajectory line were carried out deterministically using the probabilistic platform GoldSim (Golder Associates, 2002). The model of the cylindrical radial coordinate system for the engineered barriers was also developed as a deterministic model using GoldSim.

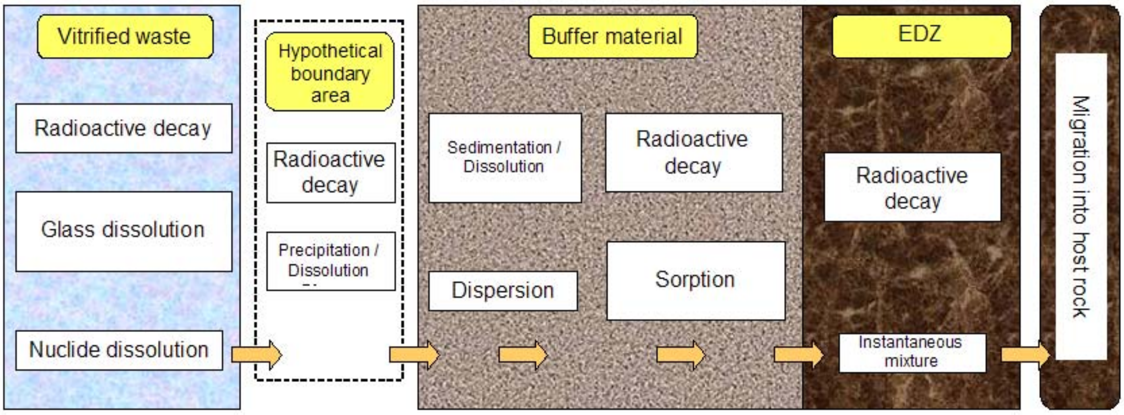


Figure 3.3.2-3 Conceptual Illustration of Mass Transport in engineered Barrier (One dimensional cylindrical radial coordinate system)

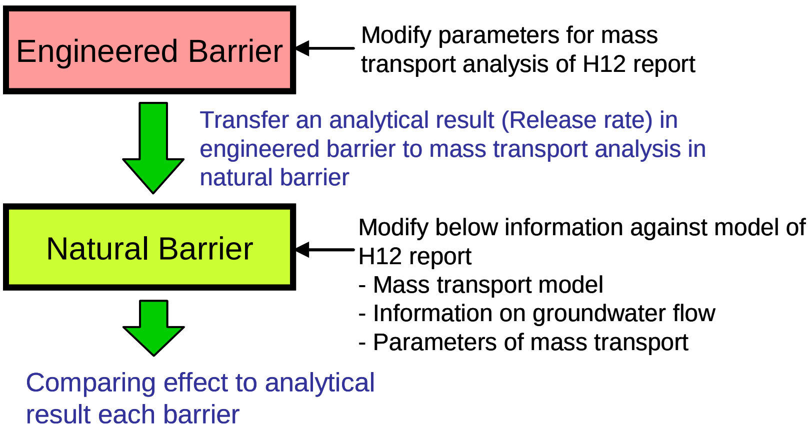


Figure 3.3.2-4 Linkage of Mass Transport Model between Engineered Barrier and Natural Barrier

Table 3.3.2-1 shows the parameters used in the mass transport analyses. Approach (a) uses the velocity and distance for each trajectory interval, whereas (b) uses velocity and distance for each trajectory interval from the Wakkanai and Koetoi Formations that has been statistically processed. The velocity obtained from the groundwater flow analysis is a Darcy velocity in which influences of pores are averaged. The true groundwater velocity is obtained by dividing it by the hydraulic advective flow porosity. The methods to determine individual parameters are described in section 3.3.3.

In the mass transport analysis for the natural barrier, the result of the mass transport analysis for the engineered barriers for one vitrified HLW (total flux out from the engineered barriers) was used as input mass into the flowpath (nuclide input source).

Table 3.3.2-1 Parameters for Mass Transport Analyses

＜Hydrogeology＞
●Actual Flow Velocity *
●Pathway Length *
*: Apply each interval data (Methodology-1), stochastic data (Methodology-2).

＜Mass Transport in Engineered Barrier＞
●Solubility in Buffer Material
●Diffusion Coefficient in Buffer Material
●Distribution Coefficient in Buffer Material
●Flow Rate into EDZ

＜Mass Transport in Natural Barrier＞
●Porosity of Host Rock
●Diffusion Coefficient of Host Rock
●Distribution Coefficient of Host Rock
●Dispersion Length
●Input Source (Amount of Mass into pathway)

(4) Nuclides considered in the mass transport analyses

Since safety assessment is not the goal of this study, not all the nuclides to be included in the safety assessment have to be considered. It is efficient to select only those nuclides that are assumed to have a dominant influence on the analysis results. Four nuclides - Se-79, Cs-135, Np-237 and Th-229 - were selected based on information from the safety assessment in the H12 Report (Table 3.3.2-2).

Table 3.3.2-2 Nuclides Considered in This Study

Radionuclide	Specification
Se-79	Dominant species at early time.
Cs-135	Dominant species which have been shown maximum value in many cases of H12 report.
Th-229	Dose peak has been shown later than Cs-135, dose is almost high next to Cs-135, in many cases of H12 report. Th-229 becomes radiation equilibrium with Np-237 in natural barrier.
Np-237	Mother radionuclide of Th-229.

3.3.3 Parameters for mass transport analysis

This section describes the methods used to determine the individual parameter values to be used in the mass transport analyses and presents the parameter values determined using these methods.

Parameters to be used in the mass transport analyses were selected as follows based on the discussion in section 3.3.2 (Table 3.3.2-1).

- True groundwater velocity and transport distance along the flowpath
- Flux through the EDZ
- Diffusion coefficient, porosity, dry density, density and distribution coefficient in the rock
- Solubility, distribution coefficient and diffusion coefficient in the buffer

Although the compositions of the groundwater and porewater in the buffer are not used directly in the mass transport analyses, they represent important input data for determining diffusion coefficients, distribution coefficients and solubilities. However, the hydrochemistry dataset and the hydrochemical model alone cannot supply sufficient information to allow the groundwater chemistry at the target depth of 450 m to be determined. Estimating groundwater and porewater chemistry using the hydrochemical model or other methods is therefore important. The estimation of groundwater and porewater chemistry will be described first and determination of other parameters will be described in sections (2) to (5). The final parameters are presented in (6).

In general, the dispersion length is about 1/10 of migration length as specified in H12 report. In this study, it was set to 0, i.e. no effect was considered in the initial stage of the study because no relevant information (including measured data) was available and it was assumed that this parameter would not have a significant effect on the release rate within the range of for groundwater flow velocity used in this study.

(1) Groundwater and porewater chemistry

(a) Groundwater chemistry

The groundwater chemistry was determined based on the results of analyses from borehole HDB-6 (approximately 600 m in length), which is the nearest borehole to the planned URL (Yamamoto et al., 2005a). Data were obtained for eleven samples taken at different depths: two samples (formation water) pumped from underground formations to the ground surface and nine samples collected by squeezing rock cores (core water). Since the core water was obtained by pressing/squeezing rock cores at 70 MPa, the properties of the core water may be different to those of the formation water due to contact with air, dissolution and/or precipitation of minerals, etc. occurring during the squeezing process (e.g. Pearson et al., 2003). The data obtained from the core water should therefore be treated with caution. Based on these data, and considering the points below, the groundwater chemistry at a depth of around 450 m in borehole HDB-6 was used as the groundwater chemistry for a depth of 450 m for the analysis domain.

- Information on depth dependence
- Results of thermodynamic analyses, and
- Information on the minerals in the Horonobe area

First, a depth profile of groundwater chemistry was prepared and thermodynamic analyses were conducted based on the data obtained from borehole HDB-6 down to a depth of 600 m. The depth profile indicates that the concentrations of Na, Cl and Mg tend to become higher with increasing depth. No depth dependence was observed for K, Ca, HCO_3/CO_3, Fe and SO_4. The results of the thermodynamic analyses indicate slight changes in the saturation index (SI = log(IAP/Ksp), where IAP is the ion activity product and Ksp is the solubility product) as a function of depth for those minerals that might dominate the groundwater chemistry. The SI values obtained for a depth of 450 m ranged from 0.4 to 0.7 for calcite and from 0 to -0.2 for amorphous silica. Based on these results, the groundwater chemistry around 450 m depth in borehole HDB-6 was estimated as follows:

- Na, Cl, Mg: No significant difference was observed between the data for formation water and core water. Therefore, a regression equation was derived from both sets of data down to 600 m depth and values around 450 m depth were calculated accordingly.

- K: No significant difference was observed between the data for formation water and core water and no significant depth dependence was observed. Therefore, the value for formation water obtained from a depth of around 450 m was used.

- Ca, HCO_3/CO_3, Si: The values for 450 m depth were estimated based on the assumption that the ion concentrations are dominated by mineral-water reactions. For amorphous silica, the SI was determined as -0.2 based on the value at a depth of around 450 m. It is assumed that the groundwater in the Horonobe area contains large amounts of dissolved gas that should be released in the pumped groundwater (degassing). The results of a thermodynamic analysis of groundwater in which carbon dioxide gas has been released indicate that the groundwater is apparently supersaturated with calcite (Nagra, 1989; Sasamoto et al., 2005). Calcite is generally known to dissolve relatively easily, even at a low temperature. Groundwater with longer residence times should have therefore be saturated with calcite (SI = 0), as assumed in the H12 Report. This assumption was taken over in the present study and the SI at 450 m depth was determined to be 0 for calcite.

- Fe, SO_4, Eh: The concentration of SO_4 was higher in the core water than in the formation water, which is assumed to be due to the oxidation-dissolution of pyrite during the squeezing process (JNC, 2005). The analysis result for the formation water should therefore be closer to the concentration of SO_4 in actual groundwater. The concentrations of Fe and SO_4 depend on the redox condition of the groundwater. There were no reliable measured data for Eh for the URL area. Since it was difficult to identify a specific reaction that would dominate the Eh in the groundwater, it was assumed that the groundwater was saturated with pyrite and siderite (SI = 0), based on general information on minerals that influence redox condition in underground formations.

The concentrations of other constituents, i.e. NH_4, PO_4, Br, Sr, Mn and Al, were determined based on the results of the analysis using formation water at around 450 m depth.

Based on the above considerations, the groundwater chemistry at a depth of 450 m in borehole HDB-6 was calculated using the geochemical calculation code PHREEQC (Parkhurst, 1995) and the thermodynamic database (JNC-TDB 011213; Yui et al., 1999) at a temperature of 25°C. The calculated values were then used for the groundwater chemistry in this study.

In order to evaluate uncertainties in the estimation of groundwater chemistry, calculations were also conducted taking into account the errors associated with the extrapolated values for Na, Cl and Mg (standard errors of the regression line) and the range of saturation indices for the minerals (SI = 0 to 1 for calcite, SI = -0.4 to 0 for amorphous silica). The result indicates that a pH ranges of 6.7 to 7.2 and the largest error in groundwater components are ±1,100 mg kg^{-1} for Na and Cl ions.. As a reference, a calculation was carried out at a temperature of 28°C, which is the estimated groundwater temperature at 450 m depth, using SPRONS.JNC (Arthur et al., 1999) to evaluate the influence of temperature; almost no difference was observed compared to the result at 25°C.

(b) Porewater chemistry

The porewater chemistry in the buffer was estimated using the same method as in the H12 Report using the estimates of groundwater chemistry described above as input data (Oda et al., 1999). The results are shown in Table 3.3.3-1.

Table 3.3.3-1 Estimated Groundwater Chemistry of 450m depth at HDB-6

TDB	JNC-TDB 011213c2.tdb		Dissolved species	[mol kg^{-1}]	[mg kg^{-1}]
Temperature [degree]	25		HCO_3^-	3.0×10^{-2}	1,823
pH	6.8		CO_3^-	2.0×10^{-5}	1.2
Pe	-2.8		SO_4^{2-}	4.4×10^{-6}	4.0×10^{-1}
Eh [mV]	-166		N_2 2)	-	-
Ion intensity	2.6×10^{-1}		NH_4^+	7.8×10^{-3}	140
Charge balance [eq] 1)	6.8×10^{-15}		Fe^{2+}	1.5×10^{-5}	0.9
	[mol kg^{-1}]	[mg kg^{-1}]	H_4SiO_4(aq)	1.1×10^{-3}	109
Na	2.3×10^{-1}	5,239	HPO_4^{2-}	5.1×10^{-7}	4.0×10^{-2}
K	2.1×10^{-3}	81	PO_4^{3-}	7.6×10^{-12}	6.0×10^{-7}
Mg	5.8×10^{-3}	141			
Cl	2.2×10^{-1}	7,806	Saturation index		
Br	5.6×10^{-4}	45	Calcite	0.0	
Ca	2.1×10^{-3}	84	Amorphous silica	-0.2	
C	4.1×10^{-2}	492	Siderite	0.0	
S	6.3×10^{-6}	0.2	Pyrite	0.0	
F	1.1×10^{-5}	0.2			
Fe	1.5×10^{-5}	0.9			
Si	1.1×10^{-3}	32			
N	7.8×10^{-3}	109			
P	1.3×10^{-6}	3.9×10^{-2}			
Sr	3.0×10^{-5}	2.6			
Mn	3.6×10^{-7}	2.0×10^{-2}			
Al	3.7×10^{-7}	1.0×10^{-2}			

1) Charge balance was adjusted by Cl^-.
2) N_2 was not considered as dissolved species in the calculation because the measured dominant component regarding to N in the Horonobe groundwater was NH_4^+.

(2) True groundwater velocity and transport distance along the flowpath and groundwater flow rate through the EDZ

(a) True groundwater velocity along the flowpath

The True groundwater velocities in the flowpath can be obtained by dividing the Darcy velocities along the streamline obtained from the groundwater flow analysis (Kurikami et al., 2005) by the hydraulic advective flow porosity.

The Darcy velocities and transport distances used in this analysis were obtained from groundwater flow analyses conducted for eighteen cases as described in section 3.2.2 (3) (Figure 3.2.2-11). The results showed a lower velocity and shorter distance in the Wakkanai Formation compared to the Koetoi Formation (Figure 3.3.3-1).

For the mass transport analysis according to approach (a) in section 3.3.2, the Darcy velocity and transport distance for each trajectory interval for the cases shown below were used in the analysis (Figure 3.3.3-1).

- Case 003: Results of the analysis and the measured results showed the best consistency.
- Case 010: Highest Darcy velocity in the Koetoi Formation (one order of magnitude higher than that of Case 003; in the Wakkanai Formation, the velocity is similar to that of Case 003).
- Case 014: Highest Darcy velocity in the Wakkanai Formation (one order of magnitude higher than that of Case 003; in the Koetoi Formation, the velocity is similar to that of Case 003).
- Case 013: Darcy velocity in the Wakkanai Formation changes from higher to lower compared to that of Case 003 with the increase in transport distance (in the Koetoi Formation, the velocity is similar to that of Case 003).

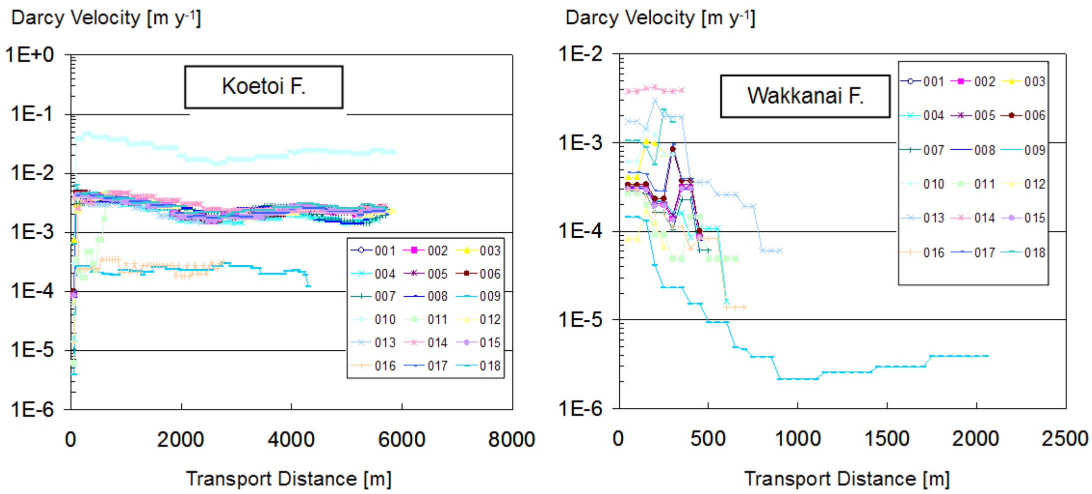


Figure 3.3.3-1 Darcy Velocity and Transport Distance Calculated from Groundwater Flow Analysis

For the analyses using approach (b) in section 3.3.2, the following parameters were determined by the average Darcy velocities and transport distances for Case 003 for the Wakkanai Formation and Koetoi Formation, respectively. In addition, the range of uncertainty was determined considering the variability of analysis results for the other cases.

Darcy velocity:

Wakkanai Formation: 6×10^{-4} m y^{-1} (uncertainty range: between 1/10 and 10 times the calculated value)

Koetoi Formation: 3×10^{-3} m y^{-1} (uncertainty range: between 1/10 and 10 times the calculated value)

Transport distance:

Wakkanai Formation: 250 m (uncertainty range: 250 - 2,000 m)

Koetoi Formation: 6,000 m (uncertainty range: 700 - 6,000 m)

For the hydraulic advective flow porosity, since no measured data using tracers or other tests were available, the results of geophysical logging conducted as part of the borehole investigations were used, with measured data for core samples as a reference (Table 3.3.3-2). Representative porosities were obtained based on the results of geophysical logging, including acoustic wave, neutron, density, natural gamma ray and x-y caliper logging (e.g. Yamamoto et al., 2005a). Figure 3.3.3-2 shows the porosities in boreholes HDB-1, 3 to 6 and 8 with the depth from the Wakkanai-Koetoi Formation boundary as a parameter. Note that the data for boreholes HDB-2 and HDB-7 were not used in this calculation because the location of the former is distant from the planned URL site and the latter is located mainly in the Yuchi Formation. The average porosity was 0.38 for the Wakkanai Formation and 0.54 for the Koetoi Formation and these values were assumed as hydraulic advective flow porosity for each formation.

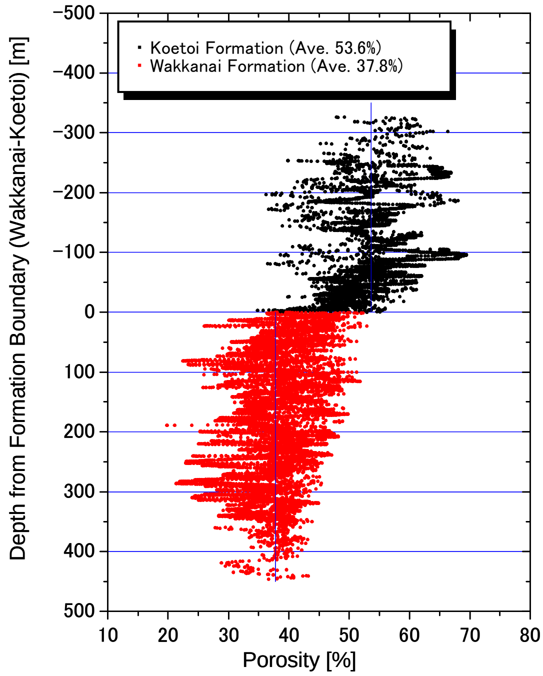


Figure 3.3.3-2 Correlation of Rock Porosity and Depth from Formation Boundary

Table 3.3.3-2 Porosity of Host Rock

	Estimated by geophysical logging data	Experimental results by core spesimen
Koetoi F.	0.54	0.52 ~ 0.60[*1]
Wakkanai F.	0.38	0.33 ~ 0.41[*2]

[Value from Literature]

Neogene sedimentary rock	0.001 ~ 0.72[*3]	
	0.02 ~ 0.41[*3]	below GL.-500m
	0.0 ~ 0.06[*4]	argillaceous / tuffaceous rock

*1: Shimo, Kumamoto, 2004. *2: Shimo et al., 2003.
*3: Sato et al., 1992. *4: Sato et al., 1999.

(b) Transport distance along the flowpath

The transport distance along the flowpath was determined as the length of the trajectory line, based on the streamlines obtained from the groundwater flow analysis (Figure 3.3.3-1).

(c) Flux through the EDZ

The flux through the EDZ means the groundwater flow rate through EDZ around the buffer, which is used as the external boundary condition in the mass transport analysis for the engineered barriers. Since the details of the disposal tunnel layout were not considered in this study, the value was determined by multiplying the Darcy velocity at the starting-point of the streamlines obtained from the groundwater flow analysis by the estimated cross-sectional area of the disposal tunnel.

The cross-sectional area of the disposal tunnel was calculated as approximately 13 m^2 based on the dimensions specified in the H12 Report for vertical emplacement (2.22 m in diameter, 4.13 m in length), and 0.5m for the EDZ thickness.

Figure 3.3.3-3 shows the flux through the EDZ calculated for each case of the groundwater flow analysis. The flux in Case 003, 0.005 $m^3\ y^{-1}$, was selected as the value for this analysis because Case 003 was the one in which the analysis and measurement results showed the best match and because the results for other cases were also close to this value. In order to evaluate the range of uncertainty, the following values were also determined based on Figure 3.3.3-3.

(a) Reference flux: 0.005 m^3 y^{-1}

(b) Maximum flux: 0.05 m^3 y^{-1}

(c) Minimum flux: 0.001 m^3 y^{-1}

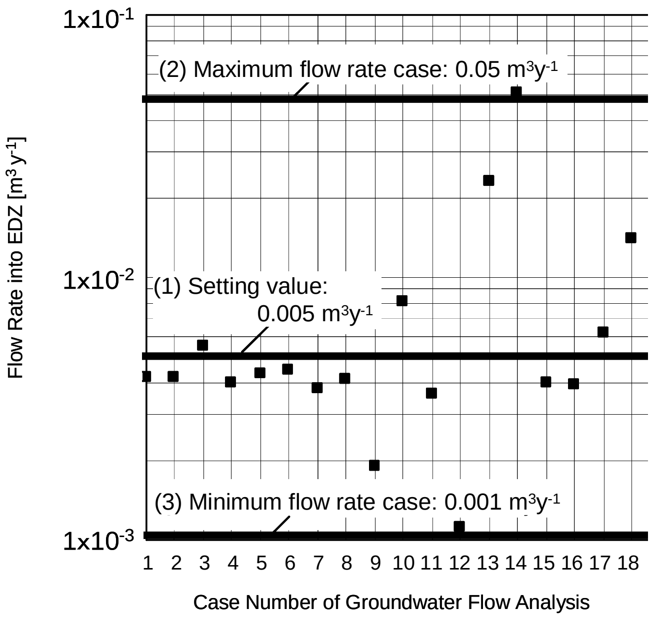


Figure 3.3.3-3 Groundwater Flow Rate through EDZ estimated from Groundwater Flow Analysis

(3) Diffusion coefficient, porosity, dry density and true density

For the diffusion coefficient, measured data for I^- were available for the Koetoi and Wakkanai Formations (Shimo et al., 2003; Shimo and Kumamoto, 2004). The parameter values for the diffusion coefficient were determined, giving preference to measured data, based on the correlation between porosity and effective diffusion coefficient (JNC, 1999a). It is also important to include distribution coefficients in the mass transport analysis for the natural barrier, which requires rock density and porosity.

The porosity and density were therefore determined first before determining the effective diffusion coefficient.

(a) Rock porosity

The porosity of the rock was determined in the same way as for the hydraulic advective flow porosity as described in section (2) above, i.e. the porosity obtained by the combined lithofacies analysis based on the results of the borehole investigations was used; the values are 0.38 for the Wakkanai Formation and 0.54 for the Koetoi Formation.

The uncertainties were set in the same range as for the hydraulic advective flow porosity, namely 1/10 of the respective values.

(b) Dry density and true density

For true density, measured data were available for the core samples from borehole HDB-6; the values are 2.31 g cm^{-3} for the Wakkanai Formation and 2.18 g cm^{-3} for the Koetoi Formation, both as an average (Yamamoto et al., 2005a). The dry densities were determined based on these values and the porosity values described above; the results are 1.43 g cm^{-3} for the Wakkanai Formation and 1.00 g cm^{-3} for the Koetoi Formation.

The uncertainty range for dry density was determined by considering the uncertainty for porosity (1/10 of the determined values); 2.22 g cm^{-3} for the Wakkanai Formation and 2.06 g cm^{-3} for the Koetoi Formation.

(c) Diffusion coefficient in rock

The effective diffusion coefficients for I^-, which were obtained from the measurements for the Koetoi Formation and Wakkanai Formation respectively, are shown in Table 3.3.3-3, together with porosities obtained by measurements on the same specimens using the water saturation method.

In order to give preference to the measured data, the average of the measured data in Table 3.3.3-3 was used. Since the average porosities of these specimens were very close to those determined in section (2) above, it was also decided to use the effective diffusion coefficients as input parameters for the analyses. The effective diffusion coefficients were corrected for temperature, as in the H12 Report. Taking the average temperature for the measured data as 25°C, they were corrected for the estimated initial temperature at 450 m depth of 28.3°C (Matsui et al., 2005), i.e. multiplied by 1.07. The results were 2×10^{-11} m^2 s^{-1} for the Wakkanai Formation and 4×10^{-10} m^2 s^{-1} for the Koetoi Formation.

Table 3.3.3-3 Effective Diffusion Coefficient and Porosity (I^-)

	Effective Diffusion Coefficient [m^2 s^{-1}]		Porosity [-]	
	Observed	Mean	Observed	Mean
Koetoi Formation	6.78×10^{-10} 3.30×10^{-10} 2.59×10^{-10} 2.08×10^{-10}	3.69×10^{-10}	0.598 0.596 0.534 0.537	0.566
Wakkanai Formation	2.34×10^{-11} 2.55×10^{-11} 8.16×10^{-12}	1.90×10^{-11}	0.414 0.384 0.332	0.377

The available volume of measured data is too small to allow discussion of the range of uncertainty and to reflect the heterogeneity of the domain to be analyzed. Therefore, the uncertainty range was determined based on the variability of the porosity by using a correlation equation between porosity and effective diffusion coefficient as follows.

- The following statistical values were used for the variability of the porosity for the Wakkanai Formation and Koetoi Formation; they were obtained from the porosity distribution described in section (2) above (Figure 3.3.3-2).

  Wakkanai Formation: Average 0.38, +2σ 0.49, -2σ 0.27

  Koetoi Formation: Average 0.54, +2σ 0.66, -2σ 0.41

- Assuming the correlation equations between porosity and diffusion coefficient for sedimentary rock (Sato, 1999), the equations that give the minimum and maximum effective diffusion coefficients were applied to the -2σ and +2σ porosities respectively. As a result, a range of 1.37×10^{-12} to 6.19×10^{-10} m^2 s^{-1} was determined for the Wakkanai Formation and 2.47×10^{-12} to 9.47×10^{-10} m^2 s^{-1} for the Koetoi Formation.

- These ranges were then corrected for the temperature of interest (from 25 to 28.3°C, multiplied by 1.07), as follows:

  Wakkanai Formation: 1×10^{-12} - 7×10^{-10} m^2 s^{-1}

  Koetoi Formation: 3×10^{-12} - 1×10^{-9} m^2 s^{-1}

In the porous medium model (section 3.3.2 (3)) used in this study, the diffusion coefficient was treated as the diffusion coefficient in porewater included in the dispersion coefficient term of equation (3.3.3-1). Therefore, for the mass transport analysis based on a porous medium model, diffusion coefficients obtained by dividing the effective diffusion coefficient by an equivalent porosity were used as input parameters.

$$D_L = \alpha_L v + D_0 \quad (3.3.3\text{-}1)$$

where D_L is the dispersion coefficient [$m^2\ s^{-1}$], α_L is the dispersion length [m], v is the pore velocity [$m\ s^{-1}$] and D_0 is the diffusion coefficient in porewater [$m^2\ s^{-1}$].

(4) Distribution coefficient in rock

The distribution coefficients for Cs and Se for sedimentary rock in the Horonobe area have been measured by batch sorption tests (Xia et al., 2004a, 2004b). For sorption of Cs on sedimentary rock, assuming that it is dominated by the illite in the rock, a model is proposed for estimating the distribution coefficient of Cs using the illite content, ion exchange constant and composition of the solution as input parameters (Bradbury and Baeyens, 2000). The application of this model to the Horonobe area is also under discussion (Xia, 2005). Considering the different levels of information available for the different elements, input parameters were determined as follows.

As the first step, using the JNC sorption database (JNC-SDB) (Suyama and Sasamoto, 2004), the measured distribution coefficient data for sedimentary rocks (mudstone, sandstone and tuff) and the minerals contained in these rocks obtained by batch sorption tests under conditions similar to those of interest were evaluated. For Cs, efforts were also made to estimate the distribution coefficients using a model taking into account the illite content in the rock and the range of ion concentrations in the groundwater that would have an influence on the sorption of Cs.

For Cs and Se, batch sorption tests have been conducted by Xia et al. (2004a, 2004b) using crushed rock samples obtained from four different depths (two samples each for the Koetoi and Wakkanai Formations) in borehole HDB-3 (Yamamoto et al., 2004a) in three different types of water, i.e., distilled water, natural groundwater obtained from the borehole and simulated groundwater. Focusing on the results of the tests using the natural and simulated groundwater, corresponding to the conditions in a disposal system (in this case lower than 1×10^{-5}mol L^{-1}), were selected as the distribution coefficients when concentration dependence was observed. No significant difference was observed in the distribution coefficients for Cs obtained by batch sorption tests between the Koetoi Formation and Wakkanai Formation. For Se, the distribution coefficient was higher for one rock sample from the Koetoi Formation than for the others. However, following the results for Cs, the same value was used for both formations. Thus, parameter values were determined based on measured data for sedimentary rocks in the Horonobe area and the range of uncertainty was determined based on the JNC-SDB.

The distribution coefficients for Np and Th were also determined from the JNC-SDB. For the uncertainty ranges, upper and lower bounding values were determined so that almost all data were included, except for extremely high or low values.

Table 3.3.3-4 shows the distribution coefficients and uncertainty ranges determined in this way. The distribution coefficient for Cs was considered based on data measured by sorption tests using rock samples from the Horonobe area, estimations using a model assuming sorption of Cs predominantly by illite and the distribution of existing distribution coefficients in the JNC-SDB. As a result, the distribution coefficient was determined based on the measured values and the uncertainty range was specified based on the estimated values from the model.

Table 3.3.3-4 Distribution Coefficient in Host Rock

Element	Setting Value [m^3 kg^{-1}]	Width of Uncertainty [m^3 kg^{-1}]
Cs	0.1 ~ 0.5	0.05 ~ 1
Se	0.01 ~ 0.1	0.001 ~ 0.1
Np	1 ~ 10	0.1 ~ 50
Th	1 ~ 10	0.1 ~ 50

(5) Diffusion coefficient, distribution coefficient and solubility in the buffer

Diffusion coefficients, distribution coefficients and solubilities in the buffer were determined for the specifications of the engineered barriers defined in the H12 Report and the porewater chemistry estimated in section (1) above (Table 3.3.3-1).

(a) Solubility

Solubilities in the porewater of the buffer were determined in the same way as in the H12 Report. The thermodynamic database of radioactive elements, the JNC-TDB, has been developed for the performance assessment of the H12 Report (Yui et al., 1999). Solubilities were calculated by using the geochemical code PHREEQC with the JNC-TDB and assuming an appropriate solubility-limiting solid phases. Solubilities calculated for Se, Np, Th in the porewater condition were shown in Table 3.3.3-1. Since Cs is soluble, no calculation was made for it.

Table 3.3.3-5 shows the determined solubilities in the porewater.

Table 3.3.3-5 Solubility

Se	Calculated	4.55E-11 mol/L
	Dominant sp.	HSe^- : 100%
	Setting Value	5.E-11 mol/L
Np	Calculated	4.93E-08 mol/L
	Dominant sp.	$Np(OH)_2(CO_3)_2^{2-}$: 93.9% $Np(OH)_4$: 6.1%
	Setting Value	5.E-08 mol/L
Th	Calculated	7.10E-06 mol/L
	Dominant sp.	$Th(OH)_3CO_3^-$: 100%
	Setting Value	8.E-06 mol/L

(b) Distribution coefficient

Distribution coefficients in the buffer were derived from the distribution coefficients for compacted bentonite, as was done in the H12 Report. The ion concentrations shown in Table 3.3.3-1 are, as descried in section (c) below, similar to those specified in the H12 Report for the porewater under saline groundwater conditions. Considering the influence of ionic strength observed for Cs in the batch sorption tests, the distribution coefficients in the H12 Report that were specified for saline groundwater conditions were taken over in this study.

Table 3.3.3-6 shows the determined distribution coefficients for the buffer.

Table 3.3.3-6 Distribution Coefficient in Buffer Material

Element	Setting Value [m^3 kg^{-1}]
Cs	0.001
Se	0
Np	1
Th	1

(c) Diffusion coefficient

The diffusion coefficients for nuclides in the buffer were basically determined in the same way as in the H12 Report, but consideration was given to the fact that the ionic strength shown in Table 3.3.3-1 (0.25) is closer to that for saline groundwater conditions (I = 0.6) in the H12 Report than for freshwater conditions (I = 0.02). Existing studies suggest the trend described below for the influence of ionic strength on the effective diffusion coefficient in compacted bentonite:

- The effective diffusion coefficients for cations show a large decrease when ionic strength reaches a level of 0.5; for an ionic strength of approximately 0.2, they remain around 3/4 of the values observed for a lower ionic strength (Suzuki, 2002).

- The effective diffusion coefficients for anions increase drastically with an ionic strength increase to 0.3 (Suzuki, 2002; Ishidera et al., 2004).

Considering that this study was in the initial stages, the diffusion coefficient for Se (anion) was determined conservatively as that for tritiated water (HTO).

The determined values were then corrected for temperature as in the H12 Report. In this study, the correction was made for the temperature in the buffer 1,000 years after disposal, which was assumed to be about 40°C (Matsui et al., 2005). The corrected values were rounded to one significant figure on the conservative side.

Table 3.3.3-7 shows the result.

Table 3.3.3-7 Effective Diffusion Coefficient in Buffer Material

	De (25 deg. C) [$m^2\ s^{-1}$]	De (offset value of 40 deg. C) [$m^2\ s^{-1}$]	Setting value [$m^2\ s^{-1}$]
Cs	2.7E−10	3.6E−10	4.E−10
Se	1.2E−10	1.6E−10	2.E−10
Np			
Th			

(6) Summary

Table 3.3.3-8 shows the parameters determined as described above. The values in bold type indicate the parameters which were highlighted in the mass transport analysis described in section 3.4.

Table 3.3.3-8 Parameters for Mass Transport Analysis

		Setting value	Width of uncertainty	Remarks
Pathway length [m]		- Methodology-1 Apply pathway information from groundwater flow analysis - Methodology-2 Wakkanai F.: 250 Koetoi F. : 6,000	- Methodology-2 Wakkanai F.: 250~2,000 Koetoi F. : 700~6,000	
Darcy velocity [m y^{-1}]		- Methodology-1 Apply pathway information from groundwater flow analysis - Methodology-2 Wakkanai F.: 6×10^{-4} Koetoi F. : 3×10^{-3}	- Methodology-2 1/10~10 times of Setting value	
Flow porosity	Wakkanai F.	0.38	1/10	
	Koetoi F.	0.54		
Dispersion length		0		
Porosity of host rock	Wakkanai F.	0.38	1/10	=Hydraulic effective porosity
	Koetoi F.	0.54		
Dry bulk density of host rock [g cm^{-3}]	Wakkanai F.	1.43	2.22	Width of uncertainty are set by 1/10 of porosity
	Koetoi F.	1.00	2.06	
Effective diffusion coefficient of host rock [m^2 s^{-1}]	Wakkanai F.	$2x10^{-11}$	$1x10^{-12}$~$7x10^{-10}$	28.3 deg. C
	Koetoi F.	$4x10^{-10}$	$3x10^{-12}$~$1x10^{-9}$	
Distribution coefficient of host rock [m^3 kg^{-1}]	Se	0.01~0.1	0.001~0.1	
	Cs	0.1~0.5	0.05~1	
	Np	1~10	0.1~50	
	Th			
Effective diffusion coefficient in buffer material [m^2 s^{-1}]	Se	$2x10^{-10}$		40 deg. C
	Cs	$4x10^{-10}$		
	Np	$2x10^{-10}$		
	Th			
Distribution coefficient in buffer material [m^3 kg^{-1}]	Se	0		
	Cs	0.001		
	Np	1		
	Th			
Solubility in buffer material [mol l^{-1}]	Se	$5x10^{-11}$		
	Cs	Soluble		
	Np	$5x10^{-8}$		
	Th	$8x10^{-6}$		
Flow rate into EDZ [m^3 y^{-1}]		0.005	0.001~0.05	

3.4 Mass transport analysis

This section describes the last two steps in the work flow (Figures 3.1.1-1 and 3.1.2-1), i.e. mass transport analysis and evaluation of the results, based on the results of the studies described in section 3.3. The detailed procedures used for the analyses, the work actually performed, the results of the analyses and their discussion are summarized below.

The discussion of the analysis results focused on the release rate from the Koetoi Formation through the Koetoi Formation and Wakkanai Formation in the target area, as described in section 3.3.2 (1).

3.4.1 Sensitivity analysis

The analyses started with approach (a) (see section 3.3.2 (3), heterogeneity of flowpath information directly incorporated in the model), in which mass transport was analyzed in terms of Darcy velocity and transport distance, using information obtained from the groundwater flow analysis for each trajectory interval along the streamlines. The sensitivity analysis cases were then determined according to approach (b) (see section 3.3.2 (3), heterogeneity of the flowpath information simplified, e.g. by statistical processing and then incorporated in the mass transport analyses) for mass transport analyses and discussion of the results.

The mass transport analyses were conducted according to approach (a) for four different cases defined in section 3.3.3 (2), i.e. Case 003, Case 010, Case 014 and Case 013, using the Darcy velocity and transport distance in each trajectory interval. Other parameter values were those discussed in section 3.3.3 for the mass transport analyses in both the engineered and natural barriers (Table 3.3.3-8) (for the rock, the lowest in the range of distribution coefficients were used).

The results indicate no significant release rate either for the Wakkanai Formation or Koetoi Formation. The only exception was the release rate obtained for the Wakkanai Formation in Case 014. This is because Case 014 has the highest velocity for the Wakkanai Formation. Even in Case 014, due to the low velocity in the Koetoi Formation, which is on the downstream side of the Wakkanai Formation, the release rate from the flowpath as a whole (release rate from the Koetoi Formation) was not significant. This means that transport behavior along the entire flowpath is dominated by the low-velocity zone within the flowpath.

It was concluded that a sensitivity analysis according to approach (b) would be more effective, rather than analyses using approach (a) at this stage, for evaluating the influence of the different properties of the geological structures.

As noted in section 3.3, the volume of information available in this study was too small to allow selection of a well-justified parameter value for each geological environment condition or mass transport property for the Horonobe URL area. Therefore, for setting the sensitivity analysis cases, in addition to the parameters determined as in section 3.3.3, uncertainty ranges were also taken into account (Table 3.3.3-8). However, combination of parameters including uncertainties would result in a huge number of cases and, for reasons of efficiency, uncertainty was considered only for the following seven parameters that were assumed to have a more significant effect on mass transport behavior. For other parameters, the determined values were used.

- Darcy velocity
- Flux through the EDZ (to be varied in proportion to the change in Darcy velocity)
- Transport distance
- Hydraulic advective flow porosity
- Porosity (same value as the hydraulic advective flow porosity)
- Dry density (to be varied according to porosity)
- Distribution coefficient in rock

When a change in the hydraulic advective flow porosity is assumed, a change in the porewater velocity (= Darcy velocity / hydraulic advective flow porosity) as well as changes in the porosity and thus the dry density on the retardation factor R, expressed by the following equation, may occur. A case for evaluating these effects was therefore included in the analysis cases.

$$R = 1 + \frac{\rho \cdot K_d}{\theta} \quad (3.4.1\text{-}1)$$

where R is the retardation factor [-], K_d is the distribution coefficient [m^3 kg^{-1}], ρ is the dry density [kg m^{-1}] ($=(1-\theta)$ ×actual density) and θ is porosity [-].

Determination of analysis cases and discussion of the results of the groundwater flow analyses were conducted independently for the Wakkanai Formation and Koetoi Formation, because these formations were handled separately in the groundwater flow analyses and mass transport parameters were also determined taking the differences between them into account.

The five sensitivity analysis cases as determined above are shown in Table 3.4.1-1. The true groundwater velocity was obtained by dividing the Darcy velocity by the hydraulic advective flow porosity. For parameters other than those shown in the table, the values determined in the previous sections were used. The defined cases represent the following conditions:

Case 1: Reference case for the sensitivity analyses (determined values were used for the seven selected parameters).

Case 2: For Case 1, uncertainty of Darcy velocity in the Wakkanai and Koetoi formations was considered (10 times higher than the determined values).

Case 3: For Case 2, uncertainty in the transport distance in the Koetoi Formation was considered (shorter transport distance than the determined value).

Case 4: For Case 3, uncertainties in the hydraulic advective flow porosities and porosities of both formations were considered (1/10 of the determined values).

Case 5: For Case 4, uncertainties in the distribution coefficients of both formations were considered (upper limits of the determined ranges).

The mass transport analyses were conducted first for the engineered barriers, considering the flux through the EDZ defined for Cases 1 to 5, and then for the natural barrier, considering all the parameters defined for Cases 1 to 5.

3.4.2 Discussion and conclusions

(1) Nuclide transport in the engineered barriers

The results of the mass transport analyses for the engineered barriers (temporal changes in the release rate from the engineered barriers for a flux through EDZ of 0.005 $m^3\ y^{-1}$) are shown in Figure 3.4.2-1.

The release rate of Se-79 (lower sorption on the buffer material) increases in the early stages, followed by Cs-135. The release rates of highly sorbing nuclides such as Np-237 and Th-229 reach their maximum values later. This trend is the same as that in the H12 Report.

These release rates from the engineered barriers were used as the input source for the natural barrier analysis.

(2) Results of the sensitivity analysis

Figure 3.4.2-2 shows the decrease in the release rates through the natural barrier (rate of decrease = log (maximum release rate in the natural barrier / maximum release rate in the engineered barriers)). Figure 3.4.2-3 shows temporal changes in the release rates.

Differences in the rate of decrease observed in the analysis cases are summarized below, with the focus on Cs-135 and Se-79 for which differences in decrease rate are clearly observed in Figures 3.4.2-2 and 3.4.2-3. Since Np-237 and Th-229, that are at radioactive equilibrium, are more highly sorbed than Cs-135 and Se-79 and therefore subject to greater retardation, differences in their decrease rates may not always be apparent in the figures. It is nevertheless assumed that they basically show the same trend as that for Cs-135 and Se-79.

In the reference case (Case 1), rates of decrease were very large. Compared to Case 1, in Case 2 in which the Darcy velocity (and accordingly the true groundwater velocity) was increased, the decrease rates were clearly lower (i.e. the release rates from the natural barrier were significantly higher). This can be explained by the shift in the time when the release rates from the natural barrier reach the maximum. In Case 1, it took much longer for the release rates to reach their maximum than the half-life of the nuclides and, during this period of time, radioactive decay would change significantly depending on the change in the time when the maximum release occurs (the later the occurrence of maximum release, the more nuclides decay, and vice versa). This time difference contributes to the difference in the decrease rates.

The difference between Case 2 and Case 3 can be explained in the same way, i.e. the release rates in Case 3, in which the transport distance is shorter than in Case 2, reach their maximum values earlier than in Case 2, resulting in a smaller decrease than in Case 2.

In Case 4, in which the hydraulic advective flow porosity and the porosity were reduced compared to Case 3, the decrease rates were higher than in Case 3. Looking at the relationship between the change in porosity and the change in retardation factor, according to equation 3.4.1-1, if the dependency of dry density on porosity is ignored, the decrease in hydraulic advective flow porosity and decrease in porosity cancel each other out. An increased rate of reduction in Case 4 compared to Case 3 would be due to the contribution of increased retardation factor resulting from porosity dependence of the dry density.

In Case 5, in which the distribution coefficients were increased compared to Case 4, the decrease rates are clearly higher than those for Case 4 and are between those for Cases 1 and 2. This is because the retardation factors were increased due to the increased distribution coefficients, resulting in a delayed occurrence of the maximum release rates.

Case 1 shows very large rates of decrease because it took longer for the release rates to reach their maximum than the half-life of the nuclides and the nuclides have therefore decayed to a large extent. However, this does not necessarily mean that the rate of decrease by the natural barrier is always large under conditions specified in Case 1. On the contrary, it should be recognized that the effects of the natural barrier are sensitive to the change in conditions (e.g. pore velocity, porosity and distribution coefficient), as seen in the comparisons between Cases 1 to 5.

Table 3.4.1-1 Cases of Sensitive Analysis

<table>
<tr><th>Case</th><th>Darcy Velocity
[m y^{-1}]</th><th>Flow Rate into EDZ [1)]
[m^3 y^{-1}]</th><th>Pathway Length
[m]</th><th>Hydraulic Effective Porosity
[−]</th><th>Porosity [2)]
[−]</th><th>Dry Density [3)]
[kg m^{-3}]</th><th>Distribution Coefficient of Host Rock
[m^3 kg^{-1}]</th></tr>
<tr><td>1</td><td>Wakkanai : 6×10^{-4}
Koetoi : 3×10^{-3}</td><td>0.005</td><td rowspan="2">Wakkanai : 250
Koetoi : 6,000</td><td colspan="2" rowspan="3">Wakkanai : 0.38
Koetoi : 0.54</td><td rowspan="3">Wakkanai : 1,430
Koetoi : 1,000</td><td rowspan="4">Np, Th : 1
Se : 0.01
Cs : 0.1</td></tr>
<tr><td>2</td><td rowspan="4">Wakkanai : 6×10^{-3}
Koetoi : 3×10^{-2}</td><td rowspan="4">0.05</td></tr>
<tr><td>3</td><td rowspan="3">Wakkanai : 250
Koetoi : 700</td></tr>
<tr><td>4</td><td colspan="2" rowspan="2">Wakkanai : 0.038
Koetoi : 0.054</td><td rowspan="2">Wakkanai : 2,220
Koetoi : 2,060</td></tr>
<tr><td>5</td><td>Np, Th : 10
Se : 0.1
Cs : 0.5</td></tr>
</table>

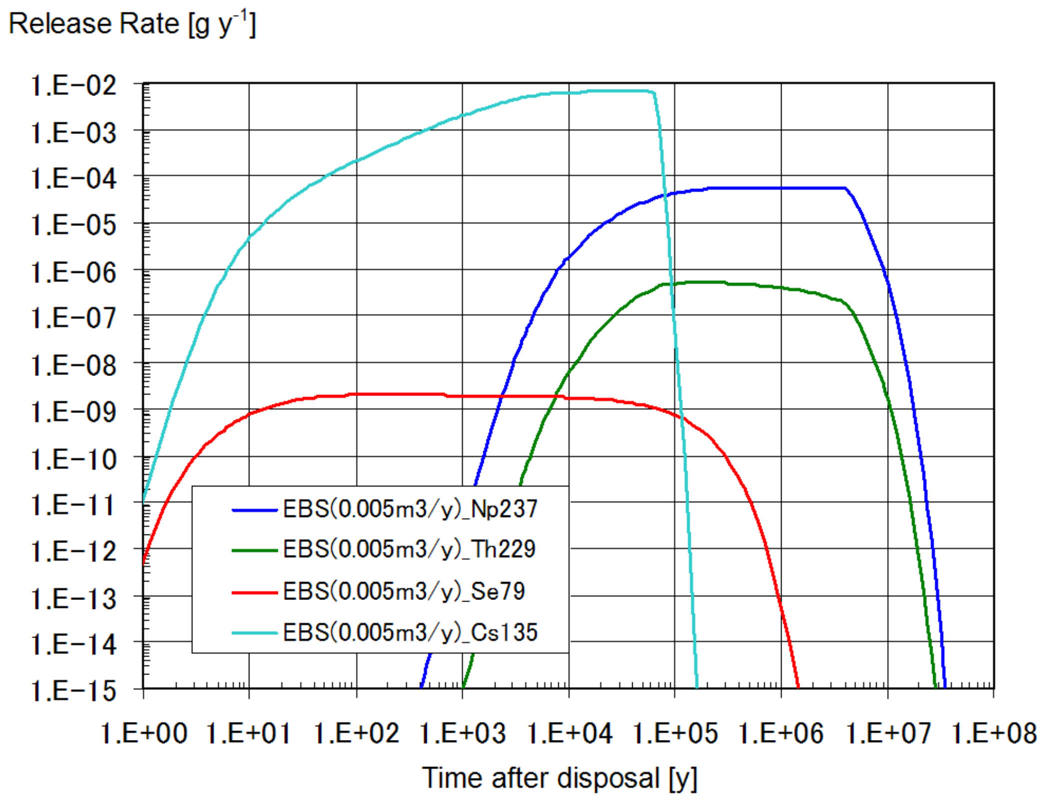
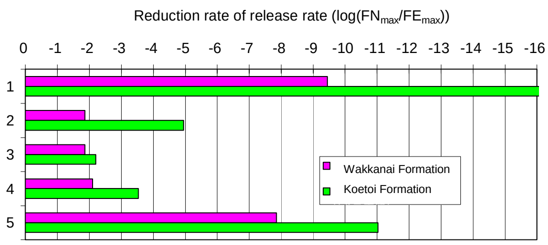


Figure 3.4.2-1 Numerical Results of Mass Transport Analysis in Engineered Barrier

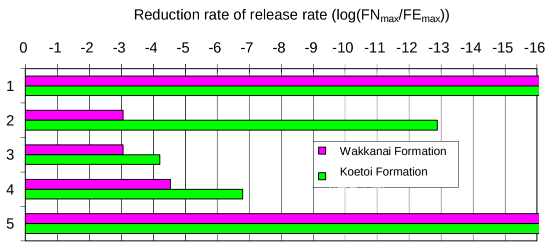
Cs-135

Case	Darcy Velocity [m y^{-1}]	Flow Rate into EDZ [1] [m^3 y^{-1}]	Pathway Length [m]	Hydraulic Effective Porosity [-]	Porosity [2] [-]	Dry Density [3] [kg m^{-3}]	Distribution Coefficient of Host Rock [m^3 kg^{-1}]
1	Wakkanai: 6x10^{-4} Koetoi : 3x10^{-3}	0.005	Wakkanai: 250 Koetoi :6,000	Wakkanai: 0.38 Koetoi : 0.54		Wakkanai: 1,430 Koetoi : 1,000	0.1
2	Wakkanai: 6x10^{-3} Koetoi : 3x10^{-2}	0.05					
3			Wakkanai: 250 Koetoi : 700				
4				Wakkanai: 0.038 Koetoi : 0.054		Wakkanai: 2,220 Koetoi :2,060	
5							0.5



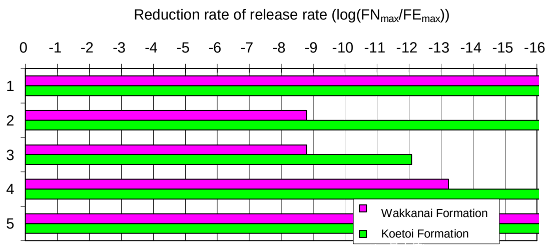
Se-79

Case	Darcy Velocity [m y^{-1}]	Flow Rate into EDZ [1] [m^3 y^{-1}]	Pathway Length [m]	Hydraulic Effective Porosity [-]	Porosity [2] [-]	Dry Density [3] [kg m^{-3}]	Distribution Coefficient of Host Rock [m^3 kg^{-1}]
1	Wakkanai: 6x10^{-4} Koetoi : 3x10^{-3}	0.005	Wakkanai: 250 Koetoi :6,000	Wakkanai: 0.38 Koetoi : 0.54		Wakkanai: 1,430 Koetoi : 1,000	0.01
2	Wakkanai: 6x10^{-3} Koetoi : 3x10^{-2}	0.05					
3			Wakkanai: 250 Koetoi : 700				
4				Wakkanai: 0.038 Koetoi : 0.054		Wakkanai: 2,220 Koetoi :2,060	
5							0.1



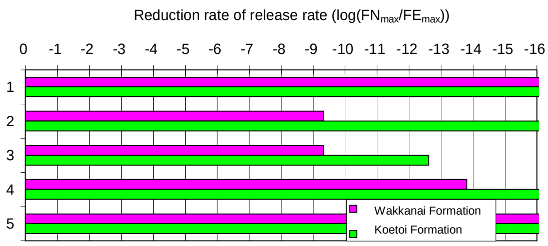
Np-237

Case	Darcy Velocity [m y^{-1}]	Flow Rate into EDZ [1] [m^3 y^{-1}]	Pathway Length [m]	Hydraulic Effective Porosity [-]	Porosity [2] [-]	Dry Density [3] [kg m^{-3}]	Distribution Coefficient of Host Rock [m^3 kg^{-1}]
1	Wakkanai: 6x10^{-4} Koetoi : 3x10^{-3}	0.005	Wakkanai: 250 Koetoi :6,000	Wakkanai: 0.38 Koetoi : 0.54		Wakkanai: 1,430 Koetoi : 1,000	1
2	Wakkanai: 6x10^{-3} Koetoi : 3x10^{-2}	0.05					
3			Wakkanai: 250 Koetoi : 700				
4				Wakkanai: 0.038 Koetoi : 0.054		Wakkanai: 2,220 Koetoi :2,060	
5							10



Th-229

Case	Darcy Velocity [m y^{-1}]	Flow Rate into EDZ [1] [m^3 y^{-1}]	Pathway Length [m]	Hydraulic Effective Porosity [-]	Porosity [2] [-]	Dry Density [3] [kg m^{-3}]	Distribution Coefficient of Host Rock [m^3 kg^{-1}]
1	Wakkanai: 6x10^{-4} Koetoi : 3x10^{-3}	0.005	Wakkanai: 250 Koetoi :6,000	Wakkanai: 0.38 Koetoi : 0.54		Wakkanai: 1,430 Koetoi : 1,000	1
2	Wakkanai: 6x10^{-3} Koetoi : 3x10^{-2}	0.05					
3			Wakkanai: 250 Koetoi : 700				
4				Wakkanai: 0.038 Koetoi : 0.054		Wakkanai: 2,220 Koetoi :2,060	
5							10



FN_{max}: Maximum of release rate outside natural barrier
FE_{max}: Maximum of release rate outside engineered barrier

Figure 3.4.2-2 Comparison of Release Rate in each Sensitivity Analysis Case

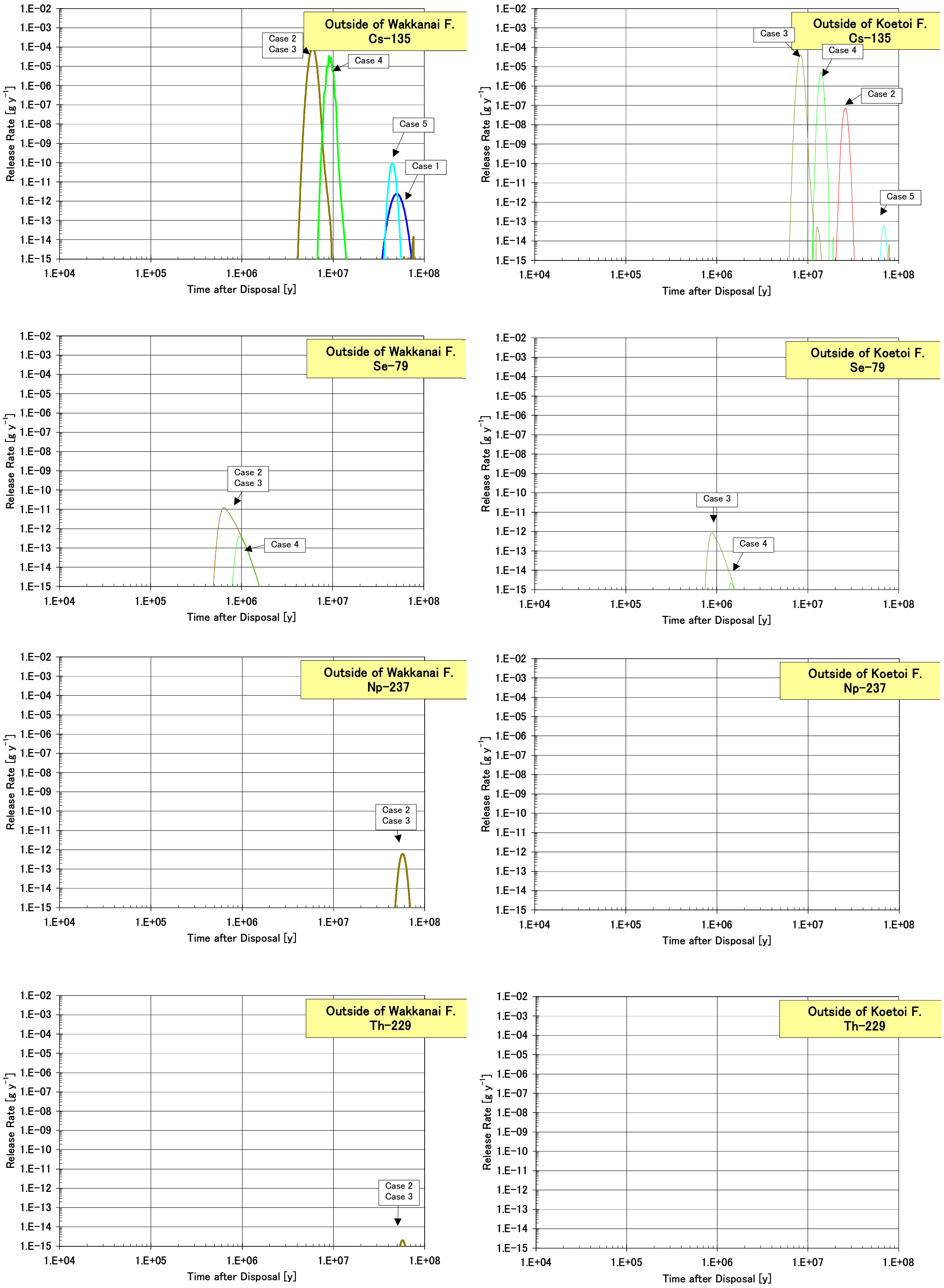


Figure 3.4.2-3 Comparison of Release Rate with Time in each Sensitivity Analysis Case

3.5 Confirming the applicability of the safety assessment methodology in the H12 Report to the surface-based investigation phase of the Horonobe URL project

This section summarizes the results of a study on the applicability of the methodologies used in the H12 Report. Based on the concept and methodology for the safety assessment in the H12 Report, a detailed work flow was formulated for conducting mass transport analyses for the sedimentary rock in the Horonobe area as a case study. Through these mass transport analyses, it was confirmed that a safety assessment methodology consisting of investigation, analysis and evaluation processes could be applied to actual geological environment conditions.

It should be noted that, although the mass transport analyses were based on a porous medium model in this study, other features such as fracture zones and faults may have a large influence not only on determination of parameter values but also on the development of concepts and models for mass transport. The work flow should therefore be revised appropriately with the progress of characterization of the geological environment conditions, allowing factors with a major influence to be identified as early as possible and the information required to evaluate such influences to be compiled. It is also important to reflect this in the relevant engineering technologies, such as design processes.

How the parameter values for the mass transport analyses were determined is summarized below. The groundwater chemistry at the target depth assumed as the starting-point for mass transport was estimated based on information obtained from borehole investigations. The groundwater chemistry, mineral composition and depth dependence thereof were determined using the same method in the H12 Report. For the groundwater chemistry and the porewater chemistry, distribution coefficients, diffusion coefficients and solubilities were then determined as follows.

- The distribution coefficients in rock were determined using methods and procedures referring to those in the H12 Report, reflecting the updated sorption database and the results of the batch sorption tests using core samples from the Horonobe area.

- The diffusion coefficients were determined using methods and procedures referring to those in the H12 Report, reflecting the correlations between diffusion coefficients and porosities formulated based on the diffusion database of the H12 Report and the results of diffusion tests using core samples from the Horonobe area.

- For the solubilities and the diffusion coefficients and distribution coefficients in the buffer, it was concluded that the values determined in the H12 Report required no adjustment at this stage and these were adopted as the parameter values for this study.

Considering that the available data are insufficient for determining parameter values, uncertainties in the parameters were expressed as a variation range. In addition, uncertainty in the analyses results upstream in the work flow was considered when determining input parameters for the analyses on the downstream side. In order to determine the distribution coefficients in rock, for example, uncertainty in the estimated groundwater chemistry, which is upstream, was evaluated and the upper and lower limits of the range of variation were determined by referring to the range that occurs most frequently in the histogram of the sorption database.

The porosity (or hydraulic advective flow porosity), which has proved in this study to be uncertain and to have a significant influence on the analysis results, simultaneously causes two opposing effects, i.e. a lower porosity leads to a higher true groundwater velocity and therefore to less of a decrease in the release rate while, at the same time, it increases the dry density and thus also the retardation factor (Table 3.4.1-1), causing a greater decrease in the release rate. This should be incorporated into the parameter values for the porosity and the models.

In this study, it was confirmed that the decrease in release rates caused by the natural barrier would be large, but it should be noted that the decrease rate is sensitive to changes in parameters such as pore velocity, porosity and distribution coefficient. It is therefore necessary to evaluate both the decrease rates themselves and sensitivity of the decrease rate to the change in conditions. This is particularly important if the same type of analyses as carried out in this study are conducted iteratively in the safety assessment.

3.6 Development of advanced safety assessment methodologies for the construction phase of the Horonobe URL project

This section summarizes the strategies for R&D in Phase II of the Horonobe URL project, based on the results of this study.

In its mid-term program, JAEA announced its intention to pursue R&D in cooperation with other organizations, in order to develop relevant technologies that would be fundamental to both the disposal project and the safety regulations. The achievements will be compiled systematically in a knowledge base and will contribute to the development of concepts to assuring and evaluating the safety performance of the repository. JAEA will issue comprehensive reports on the activities and achievements completed during the period of the mid-term program (JAEA, 2006). For this purpose, development of a knowledge management system has started in order to compile experience and know-how obtained in the course of the studies systematically in the knowledge base, which will allow appropriate transfer of the information required by the disposal project and the safety regulators, as well as by other stakeholders (Umeki et al., 2006). The Coordination Council for R&D on Geological Disposal, organized to ensure smooth promotion of fundamental R&D activities by the Agency for Natural Resources and Energy and JAEA, suggested a role-sharing setup and formulation of a framework for consolidating individual achievements through stronger cooperation and coordination among research institutes, and formulated an R&D program (R&D map) for Phase II (up to 2010). This stressed the importance of coordination among experts in different disciplines.

The geological disposal project in Japan follows a program with the key milestone of selecting detailed investigation area(s) by around 2013. Activities for the selection of preliminary investigation areas are now underway with full effort, under the direction of the Agency for Natural Resources and Energy and NUMO, the implementation organization for the HLW disposal project. For the safety regulations, the Nuclear Safety Commission has started studies under a committee on safety investigation of disposal of specified radioactive waste and has set up subcommittees on institutional requirements and environmental requirements in order to establish basic safety guidelines and environmental requirements for the selection of Detailed Investigation Areas.

The Special Committee on Nuclear Safety Research under the Nuclear Safety Commission requested in a 2005 report on safety research to be conducted by JAEA that methodologies for investigating and evaluating the geological environment should be developed as part of the safety assessment methodology in order to achieve the above targets.

Against the above background, efforts are being made by JAEA to effectively and efficiently develop a series of evaluation methods, with the objective of confirming safety assessment methodologies.

These efforts require consistency in terms of following points specified in the mid-term program of JAEA and the R&D map for Phase II.

- To develop and improve the methodologies applicable in relevant geological environments
- To produce final reports summarizing the results of the investigations in the surface-based investigation phase
- To systematically present examples of investigations during excavation and to verify the results of investigations from ground surface

With this in mind, the link between the geoscientific research in the Horonobe URL project and the R&D on the geological disposal should be improved. Nevertheless, the main purpose of the URL project continued to be geological research in the deep underground environment and development of the technical infrastructure within which investigations, analyses and evaluations can be conducted systematically. When making use of the information acquired in the URL project, the data should therefore be evaluated in terms of their universality or applicability.

Considering all the requirements above, the following goals and approaches have been defined for the development of advanced safety assessment methodologies.

- Continuation of data collection, development of models of the geological environment and hydrological and development of mass transport analyses that are based on data accumulated since April 2005.
- Quantitative analysis and evaluation of the uncertainties associated with the data on the geological environment and in the models developed using these data, as well as their influence on the results of mass transport analysis.
- Development of a comprehensive and integrated system in which individual assessment technologies can be easily implemented.

Specific R&D items required for achieving the above goals are shown below. R&D will be promoted to address these issues appropriately.

For development of a series of assessment methodologies, including investigation of the geological environment and modeling based on relevant geological environment data and hydrological/mass transport analyses, all the information obtained from the surface-based investigations (Phase I) of the Horonobe URL project will be used, including data from borehole investigations and from tests using rock and groundwater samples. One of the major objectives of the subsequent phases of the URL project which is investigations associated with tunnel excavation (construction of underground facilities) and is investigations in drifts will be verification of the results of investigations carried out during the surface-based investigation phase. Considering the above, the data from phase I will be synthesized with a view to planning specific programs for the subsequent phases. The influence on safety of the EDZ or support measures using cementitious materials, for example, will be important research items. Through case studies using the above-mentioned data and information, experience and know-how regarding application of the developed methodologies will be accumulated, allowing aspects for improvement and priorities to be identified.

With regard to uncertainties, all possible uncertain elements in the sequence from investigations through analyses and evaluation (including geological investigations, data interpretation and modeling) will be identified, and concrete methods for expressing these elements in the assessment will be developed, in order to establish comprehensive and systematic assessment methodologies. The influence of these uncertainties on mass transport analyses will be determined by case study analyses. Difference in the degree of influence depending on quality and quantity of the available data with the progress of the project will also be evaluated.

These studies should clarify what information is missing for which process and what level of knowledge can be achieved at which stage of the project. Any inadequacies found in data preparation should be classified according to their level of influence and reflected in the research program of the next phase. Using this iterative approach, developed methodologies should be evaluated in terms of applicability, and weaknesses should be clarified in order to improve reliability.

For effective, efficient and timely promotion of R&D in the field of geological disposal, information-sharing and a common awareness of current difficulties among the relevant organizations and researchers will be indispensable. For this purpose, use of a system for synthesizing information and technologies currently under development will be essential and should enable sharing of all relevant information from different fields, including data interpretation and model development.

References

Arthur, R. C., Sasamoto, H., Shibata, M., Yui, M. and Neyama, A.:“Development of Thermodynamic Databases for Geochemical Calculations”, JNC Technical Report, JNC TN8400 99-079 (1999).

Bradbury, M. H. and Baeyens, B. :“A Generalised Sorption Model for the Concentration Dependent Uptake of Caesium by Argillaceous Rocks”, Journal of Contaminant Hydrology, 42, pp.141-163 (2000).

Golder Associates:“GoldSim Contaminant Transport Module, User’s Guide”, Golder Associates Inc. (2002).

Imai, H., Yamashita, R., Amemiya, K. and Shiozaki, I.: “Study on regional groundwater flow analysis method in sedimentary rock mass”, JNC TJ1410 2001-002 (2001). (in Japanese)

Ishidera, T., Miyamoto, S. and Sato, H.: “The effect of silica content and NaNo3 Concentration on diffusions of C, CL and I in compact bentonite”, JNC TN8400 2004-001 (2004). (in Japanese)

Ishii, E. and Yasue, K.: “Characterization of fault zone by audio-frequency magneto telluric survey in the Horonobe underground research laboratory project: Three-dimensional distribution and hydrogeological property of the Omagari fault”, JNC TN5400 2005-009 (2005). (in Japanese)

Japan Atomic Energy Agency (JAEA): “Plan for meeting the midterm goal (Midterm plan) (October 1, 2005 to March 31, 2010)” (2006). (in Japanese)

Japan Nuclear Cycle Development Institute (JNC): “H12 : Project to establish the scientific and technical basis for HLW disposal in Japan”, JNC TN1400 99-020～024 (1999). (in Japanese)

Japan Nuclear Cycle Development Institute (JNC): “Plan of Deep Geological Research Laboratory (tentative name) Project”, JNC TN1410 98-002 (1998). (in Japanese)

Japan Nuclear Cycle Development Institute (JNC): “5-year R&D program for geological disposal of high-level radioactive waste”, JNC TN1440 2001-008 (2001). (in Japanese)

Japan Nuclear Cycle Development Institute (JNC): “Annual Report of Research and Development on HLW Geological Disposal in 2003”, JNC TN1400 2004-007 (2004). (in Japanese)

Japan Nuclear Cycle Development Institute (JNC): “H17: Development and management of the technical knowledge base for the geological disposal of HLW”, JNC TN1400 2005-014～016, 020 (2005). (in Japanese)

Kurikami, H., Takeuchi, R. and Seno, S.: “Groundwater flow analysis of Horonobe Underground Research Laboratory Project”, JNC TN5400 2005-003 (2005). (in Japanese)

Matsui, H., Tanai, K., Kawakami, S., Sato, T., Nishimura, M., Aoyagi, S., Fujita, T., Taniguchi, N., Kikuchi, H., Matsumoto, K. and Yui, M.: “Estimation of the applicability on the disposal technology under real geological environment”, JNC TN5400 2005-004 (2005). (in Japanese)

Nagra: Nagra Bulletin, Special Edition, December (1989).

Nuclear Safety Commission: “Nuclear safety research expected to Japan Atomic Energy Agency”, Special committee on nuclear safety research, June, 2005 (2005). (in Japanese)

Oda, C., Shibata, M. and Yui, M.: “Evaluation of porewater chemistry in the buffer material for the second progress report H12”, JNC TN8400 99-078 (1999). (in Japanese)

Parkhurst, D. L.:“User’s Guide to PHREEQC – A Computer Program for Speciation, Reaction-Path, Advective-Transport, and Inverse Geochemical Calculations”, U. S. Geological Survey, Water-Resources Investigation Report, 95-4227 (1995).

Pearson, F. J., Arcos, D., Bath, A., Boisson, J-Y., Fernandez, A. M., Gabler, H. E., Gaucher, E., Gautschi, A., Griffault, L., Hernan, P. and Waber, H. N.:“Mont Terri Project – Geochemistry of Water in the Opalinus Clay Formation at the Mont Terri Rock Laboratory”, Reports of the FOWG, Geology Series, No.5 (2003).

Sasamoto, H., Yui, M. and Arthur, R. C.: “Estimation of In-Situ Groundwater Chemistry Using Geochemical Modeling: A Test Case for Saline Type Groundwater in Argillaceous Rock”, Second International Meeting on Clays in Natural and Engineered Barriers for Radioactive Waste Confinement, Tours, France (Abstract), pp.259-260 (2005).

Sato, T., Ishimaru, T., Sugihara, K. and Shimizu, K.: “Data collection by literature survey on rock physical properties in Japan”, PNC TN7410 92-018 (1992). (in Japanese)

Sato, H.:“Diffusivity Database (DDB) for Major Rocks: Database for the Second Progress Report”, JNC TN8400 99-065 (1999).

Sato, T., Taniguchi, W. Fujita, T. and Hasegawa, H.: “Data collection by literature survey on rock physical properties in Japan (II)”, JNC TN7400 99-011 (1999). (in Japanese)

Seno, S., Takeuchi, R., Kurikami, H. and Hara, M.: “Estimation of groundwater recharge rate by water balance method in Horonobe Underground Laboratory Project－from August, 2003 to July, 2004－”, JNC TN5400 2005-005 (2005). (in Japanese)

Sibson, R. H.:“Fault rocks and fault mechanisms”, Journal of Geological Society, London, 133, pp.191-213 (1977).

Shimo, M., Yamamoto, H. and Kumamoto, S.: “Study on flow and mass transport through fractured soft sedimentary rocks”, JNC TJ8400 2003-028 (2003). (in Japanese)

Shimo, M. and Kumamoto, S.: “Study on flow and mass transport through fractured soft rocks (II)”, JNC TJ8400 2004-011 (2004). (in Japanese)

Shimo, M., Yamamoto, H., Kumamoto, S. Ono, M. and Fujiwara, Y.: “Modeling study on geological environment at Horonobe URL site”, JNC TJ5400 2004-004 (2005). (in Japanese)

Suyama, T. and Sasamoto, H.:” A renewal of the JNC-sorption database (JNC-SDB) addition of literature data published from 1998 to 2003”, JNC TN8410 2003-018 (2004). (in Japanese)

Suzuki, S.: “Study on diffusion behavior of nuclide on buffer material”, JNC TN8400 2002-006 (2002). (in Japanese)

Takeuchi, R., Seo, S. and Kurikami, H. : ” Horonobe underground research project – status of hydraulic investigations” Proceedings of 20th summer seminar on nuclear fuel cycle and environment, organized by NUCE, Atomic Energy Society of Japan (2004). (in Japanese)

Umeki, H., Ohsawa, H., Naito, M., Nakano, K. and Makino, H.: ”Fundamental concept of knowledge management system for geological disposal technology”, JAEA-Research 2006-078 (2006). (in Japanese)

Xia, X., Shibata, M., Kitamura, A. and Kamei, G.:“A systematic Study on Cesium Sorption on a Sedimentary Rock towards Reliable Safety Assessment Methodology for HLW Disposal”, Advances in Nuclear and Radiochemistry: Extended Abstracts of Papers presented at the Sixth International Conference on Nuclear and Radiochemistry (NRC-6), 29 August to 3 September 2004, Aachen, Germany, in cooperation with University of Cologne, GDCh, FECS, OECD-NEA and IAEA, edited by Syed M. Qaim and Heinz H. Coenen, pp.779-781 (2004a).

Xia, X., Shibata, M. and Kamei, G.:“Selenium sorption onto sedimentary rock under saline groundwater condition”, Proceedings of 20th summer seminar on nuclear fuel cycle and environment, organized by NUCE, Atomic Energy Society of Japan (2004b).

Xia, X. : “Study on nuclide sorption behavior onto sedimentary rock and fracture infillings under saline groundwater condition”, Brief report on research by JNC Postdoctoral fellows for 2004, JNC TN1400 2005-007, pp.59-67 (2005).

Yamamoto, T., Shimo, M., Fujiwara, Y., Hattori, H., Tadokoro, T., Iwama, H., Nago, M. and Kumamoto, S.: “HDB-1 borehole investigations in Horonobe Underground Research Center”, JNC TJ1400 2002-010 (2002). (in Japanese)

Yamamoto, T., Shimo, M., Fujiwara, Y., Hattori, H., Nago, M., Tadokoro, T. and Kuji, M.: “Borehole investigations of HDB-3 in the Horonobe Underground Research Program”, JNC TJ5420 2004-003 (2004a). (in Japanese)

Yamamoto, T., Shimo, M., Fujiwara, Y., Hattori, H., Nago, M., Tadokoro, T. and Kuji, M.: “Borehole investigations of HDB-4 in the Horonobe Underground Research Program”, JNC TJ5420 2004-004 (2004b). (in Japanese)

Yamamoto, T., Shimo, M., Fujiwara, Y., Hattori, H., Nago, M., Tadokoro, T. and Kuji, M.: “Borehole investigations of HDB-5 in the Horonobe Underground Research Program”, JNC TJ5420 2004-005 (2004c). (in Japanese)

Yamamoto, T., Shimo, M., Fujiwara, Y., Hattori, H., Nago, M. Tadokoro, T. and Nakagaki, S.: “HDB-6 borehole investigations in the Horonobe Underground Research Program”, JNC TJ5400 2005-004 (2005a).

(in Japanese)

Yamamoto, T., Shimo, M., Fujiwara, Y., Hattori, H., Nago, M. Tadokoro, T. and Nakagaki, S.: “HDB-7 borehole investigations in the Horonobe Underground Research Program”, JNC TJ5400 2005-005 (2005b). (in Japanese)

Yamamoto, T., Shimo, M., Fujiwara, Y., Hattori, H., Nago, M. Tadokoro, T. and Nakagaki, S.: “HDB-8 borehole investigations in the Horonobe Underground Research Program”, JNC TJ5400 2005-006 (2005c). (in Japanese)

Yamasaki, S., Nakatsuka, N., Ogata, N., Eki, N., Moriya, T., Takeuchi, R., Kunimaru, T. and Shirato, N.: ”Horonobe underground research project, the result on URL area selection performed along with field investigations of FY2001/2002”, JNC TN1400 2002-017 (2002). (in Japanese)

Yui, M., Azuma, J. and Shibata, M.:“JNC Thermodynamic Database for Performance Assessment of High-level Radioactive Waste Disposal System”, JNC Technical Report, JNC TN 8400 99-070 (1999).

4. Conclusions

4.1 Progress of the Phase I investigations

4.1.1 Improving the reliability of disposal technologies -demonstration of the applicability of repository design-

The case studies for the geological environment conditions in the Horonobe area included a study of the overall design flow for a repository, determination of input data for design based on geological environment conditions in the surface-based investigation stage and a design study for the disposal facilities and engineered barriers. The depth of interest for these studies was set at 450 m, considering the required mechanical stability of the disposal pits. Based on these studies, the applicability of the design methodology used in the H12 Report and potential areas for improvement were reviewed and recommended design methods were identified. The main results are summarized below.

- A practical overall design flow for the repository was proposed, which was developed with the aim of facilitating design work for the repository by reviewing the overall design flow in the H12 Report in terms of the relationship between the individual designs for the overpack, buffer and backfill and the facility design.
- Practical indicators for the evaluation of tunnel stability were proposed that take into account the tunnel stability analysis method based on standard support measures formulated by an empirical approach and intelligent construction.
- Evaluations of the long-term stability of rock under anisotropic stress conditions and long-term rock deformation during construction and operation were proposed, in addition to the evaluation of tunnel stability and design of support as conducted in the H12 Report. This is because the sedimentary rock in the Horonobe URL site has low strength, is subject to anisotropic pressure and is porous.
- When evaluating the stability of a disposal pit (without support), rock creep behavior during construction and operation should be taken into account, in addition to an elastic-perfectly plastic analysis, because disposal pits located in sedimentary rock with a low competence factor are likely to become unstable due to rock creep before emplacement of the waste packages.
- The concept for the design of engineered barriers in the H12 Report is applicable. For the evaluation of the long-term integrity of the engineered barriers, the reliability of the methodology was improved by improving models and compiling input data since the H12

report.

Important points that should be considered during the surface-based investigation phase are listed from the perspectives of investigation of the geological environment, design of the facilities, design of the engineered barriers, design of the closure system, etc. as follows:

- For the investigation of the geological environment, when materials that can be used for support in the disposal facilities are restricted due to chemical stability of concrete supports, it will be necessary to evaluate whether a geological environment that will ensure tunnel stability with support made using the selected materials is available. For the acquisition of hydrochemical data, since contact of samples with the atmosphere is unavoidable, the measured data need to be corrected based on a comprehensive consideration of the results of thermodynamic analyses and information on mineralogy. There are as yet no reliable measured data on parameters such as pH and Eh, and the data correction method has not been validated. The groundwater in the Horonobe area is assumed to originate from fossil seawater and to have a long residence time. In addition to corrections based on the chemical equilibrium theory, corrections based on gaseous content determined during degassing also need to be considered.
- The surface-based investigations will not provide sufficient knowledge for detailed clarification of the range of deep geological environments in which waste packages will be emplaced. This type of knowledge can only be acquired using a stepwise approach. Thus, the design based on the surface-based investigations should be constantly updated using the intelligent construction approach and analyses should focus on parameters that can be compared with data obtained during construction and operation. Although the double-support concept allows use of supports with reduced thickness, the concept still requires further study of its safety aspects, due to the fact that it allows rock loosening to a certain extent.
- When designing the overpack, buffer and backfill, the interactions among these individual components should be evaluated and focus placed on the factors with significant influences under the geological conditions of interest, allowing the design process to be rationalized. It will also be necessary to clarify the relationship between geological environment conditions and safety functions, e.g. the effects of extrusion and intrusion of the buffer material would not be significant under saline groundwater conditions but would be under freshwater conditions.
- With regard to the closure system design, performance of the backfill can be influenced by groundwater depending on its ion concentration. Particularly for saline groundwater conditions, more data should be collected on the self-sealing properties of backfill with a

low bentonite content.

4.1.2 Development of advanced safety assessment methodologies -confirmation of the applicability of the safety assessment methodology-

As part of the cooperation between teams responsible for geoscientific research and for R&D on geological disposal, a mass transport analysis was carried out using geological information obtained in the Horonobe URL project. The objectives of the mass transport analysis were:

(a) To perform a mass transport analysis based on the results of surface-based investigations as an example and to outline procedures for developing analysis models and determining parameters.

(b) To identify important points during investigation of the geological environment or development of analysis models to be reflected in planning the program for the next phase. This is done by studying sensitive factors or factors with a large uncertainty for the mass transport analysis.

Here, findings obtained by the mass transport analyses using the geological information (investigations and modeling of the geological environment, and groundwater flow analyses) obtained during surface-based investigation phase of the Horonobe URL project up to the end of March 2005 are summarized.

Based on the concept and methodology for the safety assessment used in the H12 Report, a detailed work flow was formulated for performing mass transport analyses for the sedimentary rock in the Horonobe area as a case study. Through the mass transport analyses conducted according to this work flow, it was shown that a series of safety assessment methodologies consisting of investigation, analysis and evaluation processes could be applied to an actual geological environment.

In developing a mass transport model, the porous medium concept was used in accordance with the assumption made when formulating the investigation program for the Horonobe URL project that groundwater would be transported through pores distributed at the grain boundaries of minerals in the rock, considering that sedimentary rocks are found in the area of interest. The results of investigations up to the end of March 2005 suggest the possibility of a fracture zone in the Wakkanai Formation functioning as a groundwater flowpath. However, since insufficient information has been acquired on the characteristics of the fracture zone, and since baseline evaluations require mass transport analyses to be conducted as investigations progress, the above porous medium model was used.

The results of the mass transport analyses indicate high retardation in both the Wakkanai Formation and Koetoi Formation. However, with reference to the distribution of hydraulic conductivities obtained in the investigations, the fracture zone is likely to function as a flowpath in the Wakkanai Formation. This likelihood will need to be checked by re-evaluating the concept for the hydrogeological model, based on knowledge to be acquired by investigations in subsequent phases and through investigations to date. Also required is development of a method for applying alternative concepts such as the dual porosity model that takes into account the effect of fractured media or rock matrix and fractures. Applying the fractured media concept will have a significant impact on the results of the mass transport analyses and an understanding of the characteristics of the fracture zone in the Wakkanai Formation as well as those in the Koetoi Formation, that are assumed to originate from tectonic movement in the same era as the Wakkanai Formation, will be important. If the role of fracture zones as pathways differs between the Wakkanai Formation and the Koetoi Formation, the difference may be used as an example of modeling of regions with different retardation effects.

Studying the applicability of safety assessment methods taking into account the effects of engineered barrier materials such as concrete and grout in and around the tunnels and the EDZ is important for establishing links between the areas of engineering and safety assessment, and for establishing a more practical evaluation system.

In the surface-based investigation phase, only rough information on the geological environment can be acquired and much information will be unavailable due to the large area to be investigated. Existing information may also need to be used for the studies. In these cases, the quality of the information (source and accuracy and data acquisition methods) should be examined in advance and used as a basis for making uncertainty ranges. In cases were there is still ambiguity, measures such as determining uncertainty ranges through discussions among experts based on empirical knowledge would be required.

By illustrating specific information flow consisting of work elements and their correlations, quality and conditions of the data and background information and assumptions for the interpretation of the data and modeling, and justification of decisions could be clarified. The approach identifying specific work items based on step-by-step discussions after sharing the framework and direction of the work among interested parties, and reviewed to improve the knowledge obtained to date iteratively at each interval had a very important role in promoting effective discussions and refining work procedures.

4.2 Future plans

4.2.1 Improvement of the reliability of disposal technologies -demonstrating the applicability of repository design-

The development of fundamental engineering technologies will involve studying the applicability of technologies to relevant geological environments and systematic synthesis of the results throughout Phase II of the Horonobe URL project. For this purpose, an R&D program for the period up to 2009 was formulated for the R&D categories of development of engineering technologies in Phase II as discussed in section 2.3, investigations using information to be obtained during the construction of the Horonobe URL or using the URL itself and laboratory tests and analytical studies for the geological environment conditions at Horonobe.

The R&D related to the construction of the underground facilities will involve demonstration of the applicability of low-pH cement for support and grouting for construction work, verification of design methods developed during the surface-based investigation phase and demonstration of construction technologies during actual construction of the underground facilities. The latter involves acquiring information required for evaluating the corrosion lifetime of the overpack and the design of the buffer in the geological environment conditions at Horonobe, developing coupled analysis methods and planning in-situ tests of long-term mechanical behavior of the buffer and rock. For the analysis of the long-term alteration of the buffer material, a model and database will be developed to analyze the influence of cementitious materials on the buffer together with the R&D group on TRU waste disposal. As part of the development of coupled thermal-hydro-mechanical-chemical analysis codes, experiments will be carried out using the geological environment data acquired in Phase II (at intermediate depth) in order to accumulate experience with application and to enhance the reliability of the codes. For research on gas migration, data on the gas permeability of rocks will be acquired by tests using rock cores obtained at Horonobe, preliminary analyses will be conducted and an in-situ test program formulated using the results.

4.2.2 Development of advanced safety assessment methodologies -confirmation of the applicability of safety assessment methodology-

For confirmation of the applicability of safety assessment methodologies, a series of methodologies related to mass transport analyses under specific geological environment conditions will be studied during Phase II of the Horonobe URL project. Iterating a series of studies as discussed in this report will allow technical findings to be compiled. For this purpose, the geological environment investigation program in Phase II and subsequent phases will be formulated taking into account issues identified in this study. In particular, using samples of rocks and groundwater to be collected from the tunnels, data on groundwater flow and mass transport parameters shall be acquired and accumulated to enhance understanding of phenomena and refine analysis models. A unique problem for areas with sedimentary rocks is that the use of materials such as concrete and grouting will be unavoidable and their long-term effects will have to be taken into account.

The work flow and details of specific tasks will be refined based on the quality and quantity of available geological environment information and knowledge including the information to be acquired iteratively in the Phase II of the Horonobe URL project, and an upgraded knowledge base that should form the basis of the safety assessment technologies in the surface-based investigation stage. Lessons learned will be reflected in the investigation of the geological environment, development of engineering technologies and safety assessment research.

This is a blank page.

国際単位系（SI）

表１．SI 基本単位

基本量	SI 基本単位	
	名称	記号
長さ	メートル	m
質量	キログラム	kg
時間	秒	s
電流	アンペア	A
熱力学温度	ケルビン	K
物質量	モル	mol
光度	カンデラ	cd

表２．基本単位を用いて表されるSI組立単位の例

組立量	SI 基本単位	
	名称	記号
面積	平方メートル	m^2
体積	立法メートル	m^3
速さ，速度	メートル毎秒	m/s
加速度	メートル毎秒毎秒	m/s^2
波数	毎メートル	m^{-1}
密度，質量密度	キログラム毎立方メートル	kg/m^3
面積密度	キログラム毎平方メートル	kg/m^2
比体積	立方メートル毎キログラム	m^3/kg
電流密度	アンペア毎平方メートル	A/m^2
磁界の強さ	アンペア毎メートル	A/m
量濃度[(a)]，濃度	モル毎立方メートル	mol/m^3
質量濃度	キログラム毎立法メートル	kg/m^3
輝度	カンデラ毎平方メートル	cd/m^2
屈折率[(b)]	（数字の）1	1
比透磁率[(b)]	（数字の）1	1

(a) 量濃度（amount concentration）は臨床化学の分野では物質濃度（substance concentration）ともよばれる。
(b) これらは無次元量あるいは次元１をもつ量であるが、そのことを表す単位記号である数字の１は通常は表記しない。

表３．固有の名称と記号で表されるSI組立単位

組立量	SI 組立単位			
	名称	記号	他のSI単位による表し方	SI基本単位による表し方
平面角	ラジアン[(b)]	rad	1[(b)]	m/m
立体角	ステラジアン[(b)]	sr[(c)]	1[(b)]	m^2/m^2
周波数	ヘルツ[(d)]	Hz		s^{-1}
力	ニュートン	N		$m\ kg\ s^{-2}$
圧力，応力	パスカル	Pa	N/m^2	$m^{-1}\ kg\ s^{-2}$
エネルギー，仕事，熱量	ジュール	J	N m	$m^2\ kg\ s^{-2}$
仕事率，工率，放射束	ワット	W	J/s	$m^2\ kg\ s^{-3}$
電荷，電気量	クーロン	C		s A
電位差（電圧），起電力	ボルト	V	W/A	$m^2\ kg\ s^{-3}\ A^{-1}$
静電容量	ファラド	F	C/V	$m^{-2}\ kg^{-1}\ s^4\ A^2$
電気抵抗	オーム	Ω	V/A	$m^2\ kg\ s^{-3}\ A^{-2}$
コンダクタンス	ジーメンス	S	A/V	$m^{-2}\ kg^{-1}\ s^3\ A^2$
磁束	ウエーバ	Wb	Vs	$m^2\ kg\ s^{-2}\ A^{-1}$
磁束密度	テスラ	T	Wb/m^2	$kg\ s^{-2}\ A^{-1}$
インダクタンス	ヘンリー	H	Wb/A	$m^2\ kg\ s^{-2}\ A^{-2}$
セルシウス温度	セルシウス度[(e)]	℃		K
光束	ルーメン	lm	cd sr[(c)]	cd
照度	ルクス	lx	lm/m^2	m^{-2} cd
放射性核種の放射能[(f)]	ベクレル[(d)]	Bq		s^{-1}
吸収線量, 比エネルギー分与, カーマ	グレイ	Gy	J/kg	$m^2\ s^{-2}$
線量当量, 周辺線量当量, 方向性線量当量, 個人線量当量	シーベルト[(g)]	Sv	J/kg	$m^2\ s^{-2}$
酸素活性	カタール	kat		s^{-1} mol

(a)SI接頭語は固有の名称と記号を持つ組立単位と組み合わせても使用できる。しかし接頭語を付した単位はもはやコヒーレントではない。
(b)ラジアンとステラジアンは数字の１に対する単位の特別な名称で、量についての情報をつたえるために使われる。実際には、使用する時には記号rad及びsrが用いられるが、習慣として組立単位としての記号である数字の１は明示されない。
(c)測光学ではステラジアンという名称と記号srを単位の表し方の中に、そのまま維持している。
(d)ヘルツは周期現象についてのみ、ベクレルは放射性核種の統計的過程についてのみ使用される。
(e)セルシウス度はケルビンの特別な名称で、セルシウス温度を表すために使用される。セルシウス度とケルビンの単位の大きさは同一である。したがって、温度差や温度間隔を表す数値はどちらの単位で表しても同じである。
(f)放射性核種の放射能（activity referred to a radionuclide）は、しばしば誤った用語で”radioactivity”と記される。
(g)単位シーベルト（PV,2002,70,205）についてはCIPM勧告2（CI-2002）を参照。

表４．単位の中に固有の名称と記号を含むSI組立単位の例

組立量	SI 組立単位		
	名称	記号	SI 基本単位による表し方
粘度	パスカル秒	Pa s	$m^{-1}\ kg\ s^{-1}$
力のモーメント	ニュートンメートル	N m	$m^2\ kg\ s^{-2}$
表面張力	ニュートン毎メートル	N/m	$kg\ s^{-2}$
角速度	ラジアン毎秒	rad/s	$m\ m^{-1}\ s^{-1}=s^{-1}$
角加速度	ラジアン毎秒毎秒	rad/s^2	$m\ m^{-1}\ s^{-2}=s^{-2}$
熱流密度，放射照度	ワット毎平方メートル	W/m^2	$kg\ s^{-3}$
熱容量，エントロピー	ジュール毎ケルビン	J/K	$m^2\ kg\ s^{-2}\ K^{-1}$
比熱容量，比エントロピー	ジュール毎キログラム毎ケルビン	J/(kg K)	$m^2\ s^{-2}\ K^{-1}$
比エネルギー	ジュール毎キログラム	J/kg	$m^2\ s^{-2}$
熱伝導率	ワット毎メートル毎ケルビン	W/(m K)	$m\ kg\ s^{-3}\ K^{-1}$
体積エネルギー	ジュール毎立方メートル	J/m^3	$m^{-1}\ kg\ s^{-2}$
電界の強さ	ボルト毎メートル	V/m	$m\ kg\ s^{-3}\ A^{-1}$
電荷密度	クーロン毎立方メートル	C/m^3	m^{-3} sA
表面電荷	クーロン毎平方メートル	C/m^2	m^{-2} sA
電束密度，電気変位	クーロン毎平方メートル	C/m^2	m^{-2} sA
誘電率	ファラド毎メートル	F/m	$m^{-3}\ kg^{-1}\ s^4\ A^2$
透磁率	ヘンリー毎メートル	H/m	$m\ kg\ s^{-2}\ A^{-2}$
モルエネルギー	ジュール毎モル	J/mol	$m^2\ kg\ s^{-2}\ mol^{-1}$
モルエントロピー, モル熱容量	ジュール毎モル毎ケルビン	J/(mol K)	$m^2\ kg\ s^{-2}\ K^{-1}\ mol^{-1}$
照射線量（Ｘ線及びγ線）	クーロン毎キログラム	C/kg	kg^{-1} sA
吸収線量率	グレイ毎秒	Gy/s	$m^2\ s^{-3}$
放射強度	ワット毎ステラジアン	W/sr	$m^4\ m^{-2}\ kg\ s^{-3}=m^2\ kg\ s^{-3}$
放射輝度	ワット毎平方メートル毎ステラジアン	$W/(m^2\ sr)$	$m^2\ m^{-2}\ kg\ s^{-3}=kg\ s^{-3}$
酵素活性濃度	カタール毎立方メートル	kat/m^3	$m^{-3}\ s^{-1}$ mol

表５．SI 接頭語

乗数	接頭語	記号	乗数	接頭語	記号
10^{24}	ヨタ	Y	10^{-1}	デシ	d
10^{21}	ゼタ	Z	10^{-2}	センチ	c
10^{18}	エクサ	E	10^{-3}	ミリ	m
10^{15}	ペタ	P	10^{-6}	マイクロ	µ
10^{12}	テラ	T	10^{-9}	ナノ	n
10^9	ギガ	G	10^{-12}	ピコ	p
10^6	メガ	M	10^{-15}	フェムト	f
10^3	キロ	k	10^{-18}	アト	a
10^2	ヘクト	h	10^{-21}	ゼプト	z
10^1	デカ	da	10^{-24}	ヨクト	y

表６．SIに属さないが、SIと併用される単位

名称	記号	SI 単位による値
分	min	1 min=60s
時	h	1h =60 min=3600 s
日	d	1 d=24 h=86 400 s
度	°	1°=(π/180) rad
分	’	1’=(1/60)°=(π/10800) rad
秒	”	1”=(1/60)’=(π/648000) rad
ヘクタール	ha	$1ha=1hm^2=10^4m^2$
リットル	L，l	$1L=1l=1dm^3=10^3cm^3=10^{-3}m^3$
トン	t	$1t=10^3$ kg

表７．SIに属さないが、SIと併用される単位で、SI単位で表される数値が実験的に得られるもの

名称	記号	SI 単位で表される数値
電子ボルト	eV	$1eV=1.602\ 176\ 53(14)\times10^{-19}J$
ダルトン	Da	$1Da=1.660\ 538\ 86(28)\times10^{-27}kg$
統一原子質量単位	u	1u=1 Da
天文単位	ua	$1ua=1.495\ 978\ 706\ 91(6)\times10^{11}m$

表８．SIに属さないが、SIと併用されるその他の単位

名称	記号	SI 単位で表される数値
バール	bar	$1\ bar=0.1MPa=100kPa=10^5Pa$
水銀柱ミリメートル	mmHg	1mmHg=133.322Pa
オングストローム	Å	$1\ Å=0.1nm=100pm=10^{-10}m$
海里	M	1 M=1852m
バーン	b	$1\ b=100fm^2=(10^{-12}cm)2=10^{-28}m^2$
ノット	kn	1 kn=(1852/3600)m/s
ネーパ	Np	SI単位との数値的な関係は、対数量の定義に依存。
ベル	B	
デジベル	dB	

表９．固有の名称をもつCGS組立単位

名称	記号	SI 単位で表される数値
エルグ	erg	$1\ erg=10^{-7}$ J
ダイン	dyn	$1\ dyn=10^{-5}N$
ポアズ	P	$1\ P=1\ dyn\ s\ cm^{-2}=0.1Pa\ s$
ストークス	St	$1\ St\ =1cm^2\ s^{-1}=10^{-4}m^2\ s^{-1}$
スチルブ	sb	$1\ sb\ =1cd\ cm^{-2}=10^4cd\ m^{-2}$
フォト	ph	$1\ ph=1cd\ sr\ cm^{-2}\ 10^4lx$
ガル	Gal	$1\ Gal\ =1cm\ s^{-2}=10^{-2}ms^{-2}$
マクスウェル	Mx	$1\ Mx = 1G\ cm^2=10^{-8}Wb$
ガウス	G	$1\ G\ =1Mx\ cm^{-2}\ =10^{-4}T$
エルステッド[(c)]	Oe	$1\ Oe \triangleq (10^3/4\pi)A\ m^{-1}$

(c) ３元系のCGS単位系とSIでは直接比較できないため、等号「 ≙ 」は対応関係を示すものである。

表10．SIに属さないその他の単位の例

名称	記号	SI 単位で表される数値
キュリー	Ci	$1\ Ci=3.7\times10^{10}Bq$
レントゲン	R	$1\ R = 2.58\times10^{-4}C/kg$
ラド	rad	$1\ rad=1cGy=10^{-2}Gy$
レム	rem	$1\ rem=1\ cSv=10^{-2}Sv$
ガンマ	γ	1 γ =1 nT=10-9T
フェルミ		1フェルミ=1 fm=10-15m
メートル系カラット		1メートル系カラット = 200 mg = 2×10-4kg
トル	Torr	1 Torr = (101 325/760) Pa
標準大気圧	atm	1 atm = 101 325 Pa
カロリー	cal	1cal=4.1858J（「15℃」カロリー），4.1868J（「IT」カロリー）4.184J（「熱化学」カロリー）
ミクロン	µ	$1\ \mu=1\mu m=10^{-6}m$

（第8版，2006年改訂）