*Sustainable and Safe Dams Around the World – Tournier, Bennett & Bibeau (Eds)*


# Design of seismic reinforcement by post-tensioned anchors in Senbon Dam

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ABSTRACT: Senbon Dam is a water supply dam with 17 meters high operated by Matsue City, which was completed in March 1918. Since this dam is an old gravity type masonry dam and the cross section is also thin, Matsue City will perform the seismic reinforcement with post-tensioned anchors in 2019 and 2020.

As for experiences of the reinforcement by post-tensioned anchors for dams in Japan, there are old cases such as the construction of the auxiliary dam in the Fujiwara dam in 1955 and the reinforcement of the rock abutment in the Kawamata arch dam in 1963. However, under the latest requirement of high durability and high capacity, it is the first domestic experience of dam body reinforcement by the post-tensioned anchor in Japan.

As technical points for applying the post-tensioned anchor to the Senbon Dam, there are following issues: "coping with a large load at narrower dam crest, construction method of the overflow part that occupies 2/3 of the dam length, solution to partial weak zones in the foundation, preservation of masonry dam landscape and others".

RÉSUMÉ: Le barrage de Senbon, achevé en 1918 avec 17 mètres de hauteur, sert à l'approvisionnement en eau et il est exploité par la ville de Matsue. Puisque ce barrage est un ancien barrage-poids en maçonnerie et que sa section transversale est également mince, un renforcement sismique sera réalisé en 2019 et 2020 avec des ancrages post-tendus.

Au Japon, en ce qui a trait aux expériences de renforcement à l'aide d'ancrages post-tendus sur un barrage ou à proximité d'un barrage, il existe des cas anciens comme la construction en 1955 du barrage auxiliaire de Fujiwara et le confortement de l'appui rocheux du barrage-voûte de Kawamata en 1963. Cependant, avec les exigences les plus récentes de durabilité élevée et de grande capacité, il s'agit de la première expérience locale au Japon de renforcement du corps d'un barrage à l'aide d'ancrages post-tendus.

Comme contraintes techniques dans l'utilisation des ancrages post-tendus au barrage de Senbon, il y a les problèmes suivants: soutenir une charge importante où la crête de barrage est la plus étroite; la méthode de construction à utiliser pour la crête déversante occupant les deux tiers de la longueur du barrage; l'approche à prendre pour les zones faibles localisées dans la fondation; la préservation de l'apparence du barrage en maçonnerie; autres contraintes.

## 1 DESIGN CONDITIONS OF POST-TENSIONED ANCHOR

### 1.1 *Geological conditions*

The foundation rock near the dam consists of mainly Paleocene granite and a partially altered zone. The foundation on which the bond length portion is placed is mostly medium-grade granite, but there is weaker fine-grade granite on the left bank.

Eight altered zones have been found in the entire foundation. The altered zones are accompanied by clay, gravel, or fragile rock, and are inclined at steep angles. In the dam axial section, the largest altered zone A-1 (width 5–8 m) is distributed at a low angle in the direction of the dam axis. These altered zones have steep angles of 60–80 degrees in the direction of downstream to upstream.

The rock mainly consists of rock grade* CM and CL. The foundation ground of the center to the left bank side is equivalent to CM–CL, which is inferior to rock grade, and continues to a depth of 50 m. The rock grade of the foundation on the right bank side is mainly CM, which is better than that of the left bank.

* Rock grade is an index of strength, hardness, crack, etc. according to rock properties. In the Senbon Dam, the bond strength is 1.0 N/mm$^2$ for CL and 1.5 N/mm$^2$ for CM.

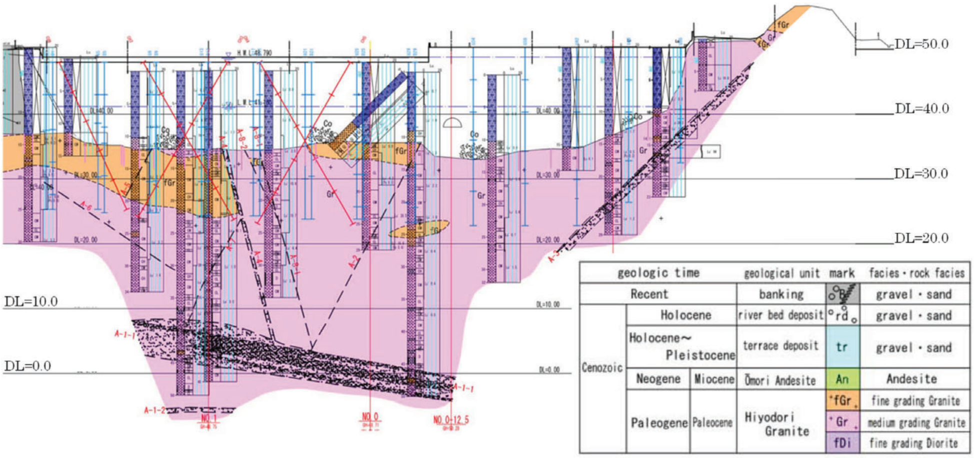



Figure 1. Geological longitudinal profile of Dam axis

## 1.2 *Dam concrete condition*

The concrete of the dam was placed about 100 years ago; its uniaxial compressive strength is 16–30 N/mm$^2$ and its unit volume weight is 20.3–23.2 kN/m$^3$, which is slightly lower than that of normal concrete. The elastic modulus is 20–57 kN/mm$^2$, which is equivalent to normal concrete.

There is no significant water leak on the dam. The Lugeon value of the dam body is as low as 0–1.9 Lu, indicating that the dam is difficult to permeate and is watertight.

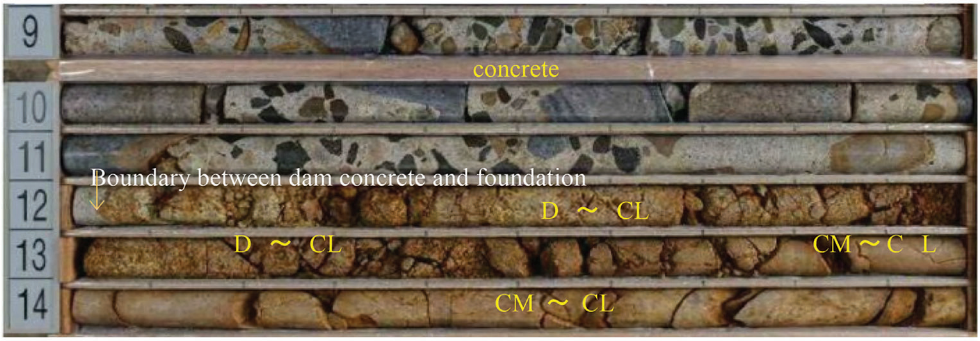
Therefore, it is evaluated that the dam concrete has sufficient strength and deformation resistance against the anchor load.

## 1.3 *Core status of dam concrete and foundation*

The situation of boring cores near the dam base is shown in Figure 2 (the positions are shown in Figure 1).

Based on the core at the boundary between the dam concrete and the foundation rock, concrete adheres tightly to the rock, and the concrete part is also good with few voids.

Boring core H28-B-1: Boundary of dam concrete and foundation is shown at 12.10m




Boring core H28-B-2: Boundary of dam concrete and foundation is shown at 10.20m

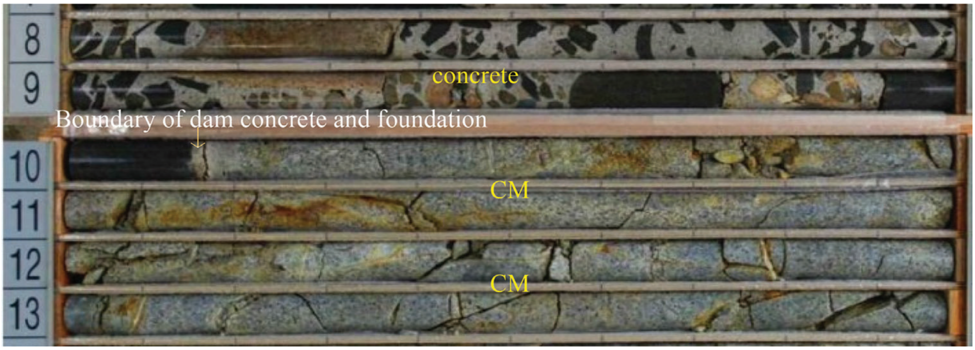



Figure 2. Situation of boring core near the dam base

## 2 STABILITY OF THE DAM

### 2.1 *Stability of the dam before reinforcement*

A stability check was made on the cross section of the overflow area and the non-overflow area. Figure 3 shows the most serious case by the stability calculations. Earthquake load 0.06g was given in the horizontal direction, when the reservoir was at the high water level.

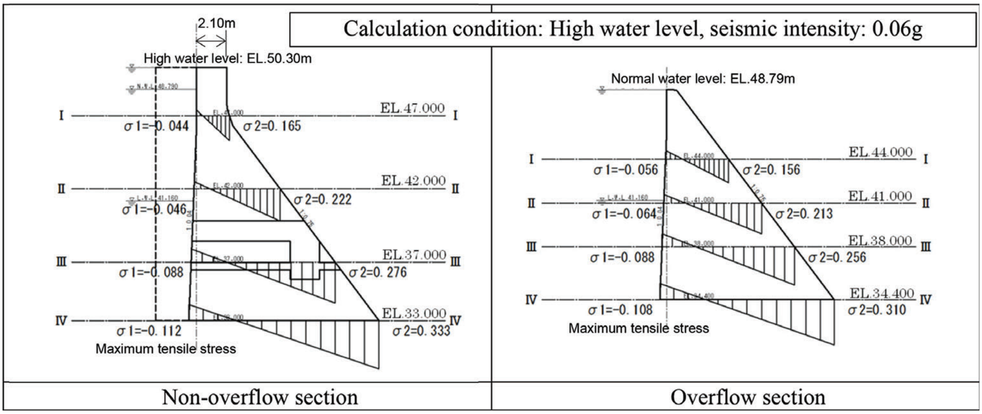



Figure 3. Result of the stability calculations: Vertical stress ($N/mm^2$) diagram before reinforcement

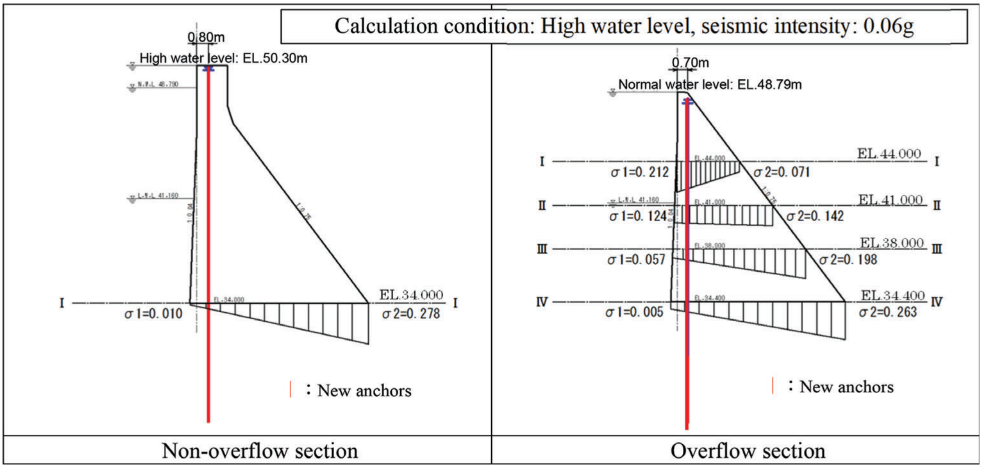



Figure 4. Vertical stress (N/mm$^2$) diagram after reinforcement by post-tensioned anchors

As for the sliding condition of the dam, the safety factor of 4 or more was secured for both the overflow section and the non-overflow section. However, as for the overturning condition, tensile stress of up to 0.112 N/mm$^2$ in the non-overflow section and up to 0.108 N/mm$^2$ in the overflow section were generated at the upstream end of the dam bottom, failing to satisfy the specification of Japanese structural regulations.

### 2.2 *Design load*

In order to ensure dam body stability, reinforcement by post-tensioned anchors was examined. The anchors are installed as necessary to ensure that tension does not occur at the upstream end of the dam. The Senbon dam anchors were positioned as close as possible to the upstream side for the stability of the dam. As a result, in the non-overflow section and the intake tower section, anchors were set at 0.8 m downstream from the dam axis, which is the upstream surface of the dam, and in the overflow section, anchors were set at 0.7 m downstream from the dam axis.

The results of the stability calculation are shown in Figure 4. The required design load per unit width was 392.4 kN/m per meter on average, but the intake tower section with the largest cross section had a higher value of 460.0 kN/m.

## 3 DESIGN OF ANCHORS AND SPECIFICATION

### 3.1 *Guidelines used for post-tensioned anchor design of the Senbon Dam*

The method of designing ground anchors in Japan is stipulated in two technical guidelines (Japanese Geotechnical Society 2012, Architectural Institute of Japan 2001). However, since these are standards for soil anchors, in this design, we preceded the design with reference to a technical research report on rock anchors (Japan Dam Engineering Center 2010), recent design experiences on rock mass post tensioned anchors in Kawamata Dam for 2015~2017, and technical recommendations (Post-Tensioning Institute 2014).

### 3.2 *Design policy*

In consideration of anchor effect, economic efficiency, safety, workability, and influence on existing countermeasures, we designed the post-tensioned anchors with the following policy.

1. Anti-corrosion specification: For double corrosion prevention, epoxy coated PC strands and fully bonded anchors were adopted.
2. Position and angle of the anchor: The anchor head was set as close as possible to the upstream end of the dam to effectively increase the compressive stress by the post-tensioned anchors. The drilling angle was set vertically so as not to affect the curtain grouting area.
3. Anchor interval in the direction of the dam axis: The anchor load was minimized since the dam top is very slender and the quality of the internal concrete can be low. For this reason, the anchor interval was based on 2.5 m so as not to overlap the existing temporary anchors (5 m interval). However, the intake tower section was arranged at intervals of 5 m to avoid the cross gallery.
4. Depth of bond length portion: From the dam base, it was decided to secure a foundation ground thickness at a depth of 5 m or more. With this configuration, the bond length portion in the foundation of CM or CL rock should be formed.
5. Bond length: The length (Lb) determined from the allowable restraining resistance of tendon (adhesion between grout and PC strands) and the length (La) determined from the allowable pull-out resistance (adhesion between grout and rock) were calculated, and the longer of Lb and La was adopted as the bond length, which means the anchorage length of the anchor system.
6. Ultimate bond strength of rock foundation: The above-mentioned La was calculated by the ultimate bond strength $\tau_u$. The $\tau_u$ values were decided mainly by referring to the shear strength of a neighboring dam, the foundation of which is the same granite as the Senbon Dam, and these values were confirmed by field rock tests. As a result, $\tau_u$ values were set as 1.0 N/mm$^2$ for CL rock and 1.5 N/mm$^2$ for CM rock. The validity of these $\tau_u$ values was confirmed by a pull-out test at the right abutment of the Senbon Dam.

### 3.3 *Deployment design of the dam post-tensioned anchors*

The depth and range of the post-tensioned anchors were determined from the rock grade distribution chart (see Figure 7), as described below.

1. Overflow section (60-m length in the dam axis direction): From the left bank to the center of the overflow area, CL rock and CM rock are distributed to the same extent at the depth of 5 m or more from the foundation surface. CL rock is dominant from the center of the overflow area to the right bank, and D rock is distributed in the altered zone. The bond length portion of anchors in the overflow area were set at CL rock ($\tau_u$ = 1.0 N/mm$^2$).
2. Non-overflow section (30-m length in the dam axis direction): CM rock is distributed throughout, but CL rock is distributed in some parts. D rock with altered zones is distributed at depths of 10–20 m from the foundation surface.

The bond length portion of anchors on the right abutment (base elevation EL 34.0 m and EL 37.0 m) were installed on CL rock with $\tau_u$ = 1.0 N/mm$^2$. As a result, the bond length portion of anchors were arranged from EL 24.0 m to 10 m below and the anchor length was about 36.0 m on average. The hole diameter of the anchor was the same as that of the overflow part, φ165 mm, and the installation pitch was set to 2.5 m.

1. Intake tower part: In the intake tower part, by increasing the circumferential length by enlarging the hole diameter to φ216 mm to increase the circumferential length, the required bond length was set to be within 10 m even in the case of CL rock.

### 3.4 *Main specifications of dam post-tensioned anchors*

Table 1 shows the calculated results according to the design policy. Fs (safety factor) of La which is calculated by $\tau_u$ is adopted as 3 according to the domestic guideline on buildings[2). $\tau_u$ means ultimate bond strength between grout and rock, and this value was determined to be

Table 1. Necessary bond length (m) by calculation result of anchors:

| Numbering of Anchors | A1-A10 A16-A23 | A11-A15 | A24-A25 | A26-A28 | A29-A34 | A35-A36 | A37-A38 |
|---|---|---|---|---|---|---|---|
| U: Perimeter around tendon (m) | 0.217 | 0.217 | 0.263 | 0.217 | 0.202 | 0.171 | 0.156 |
| La: Necessary bond length (m) to $\tau_u$ | 7.07 | 5.40 | 8.36 | 7.10 | 6.31 | 4.92 | 4.27 |
| Lba: Necessary bond length (m) to $\tau_{ba}$ | 5.63 | 5.63 | 7.20 | 5.65 | 5.40 | 4.96 | 3.86 |
| Larger value of La and Lba (m) | 7.07 | 5.63 | 8.36 | 7.10 | 6.31 | 4.96 | 4.27 |
| Round up value of bond length (m) | 7.50 | 6.00 | 8.50 | 7.50 | 6.50 | 5.00 | 4.50 |

1.0 N/mm$^2$ according to the results of the field pull-out test at Senbon Dam and the rock shear strength test at a neighboring dam.

La is also calculated by $\tau_{ba}$. $\tau_{ba}$ means ultimate bond strength between grout and PC strands, and this value was determined to be 1.0 N/mm$^2$ from the designated value by the grout compressive strength 30 N/mm$^2$ for the design standard.

Table 2 shows the results of boring diameter, fixing length, free length, etc. determined according to Table 1 and the design policy. Figures 5, 6 and 7 show the plan view, transverse and longitudinal section, respectively.

Table 2. Specification of anchors: Numbering, Design Load, Bore diameter, Bond length, etc.

| Position / specification | Overflow part | | Non-overflow part: Intake tower | | Right side | Right abutment | |
|---|---|---|---|---|---|---|---|
| Numbering of Anchor | A1-A10 A16-A23 | A11-A15 | A24-A25 | A26-A28 | A29-A34 | A35-A36 | A37-A38 |
| Number of Anchors | 18 | 5 | 2 | 3 | 6 | 2 | 2 |
| Design Load (kN) | 1,099 | | 1,702 | 1,104 | 981 | 765 | 543 |
| Lock off Load (kN) | 1,221 | | 1,891 | 1,227 | 1,090 | 850 | 603 |
| Number of strand ( φ15.2mm ) | 8 | | 11 | 8 | 7 | 5 | 4 |
| Bore diameter (mm) | 165 | 216 | | 165 | | | 135 |
| Bond length (m) | 7.5 | 9.0 | 8.5 | 7.5 | 6.5-7.5 | 6.5-7.0 | 4.5 |
| Free length (m) | 16.0-19.0 | 18.5-19.0 | 21.5-22.0 | 22.0 | 18.5-22.0 | 16.5-17.5 | 11.5-13.5 |
| Anchor length (m) | 23.5-26.5 | 27.5-28.0 | 30.0-30.5 | 29.5 | 26.0-28.5 | 23.5-24.0 | 16.0-18.0 |
| Rock grade of Bond length | CL-CM | CL-CM-D | CL-CM | CL-CM | CL-CM-D | CL-CM-D | CL-CM-D |
| Pitch of anchors(m) | 2.5 | | 3.9 | 2.0 | 2.5 | | 2.0 |

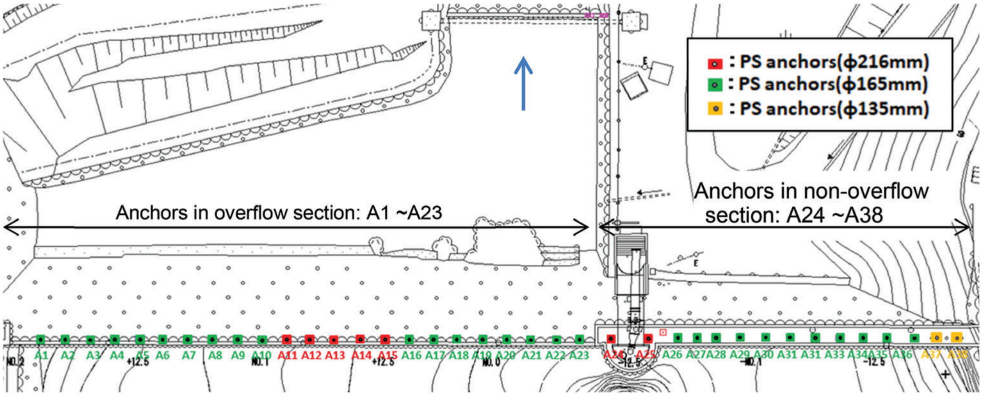



Figure 5. Plan view of Senbon Dam on layout of post-tensioned anchors

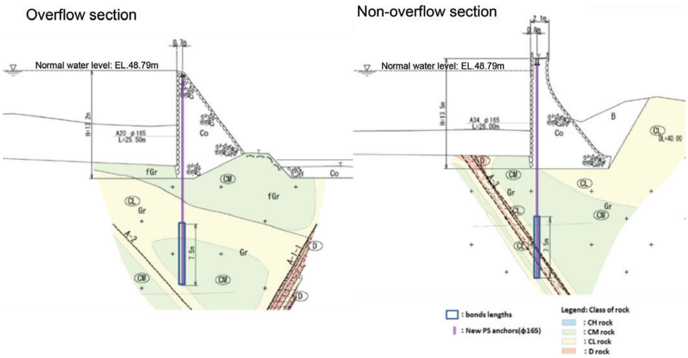



Figure 6. Cross section profile on layout of post-tensioned anchors

## 4 TWO-DIMENSIONAL ANALYSIS

### 4.1 *Outline*

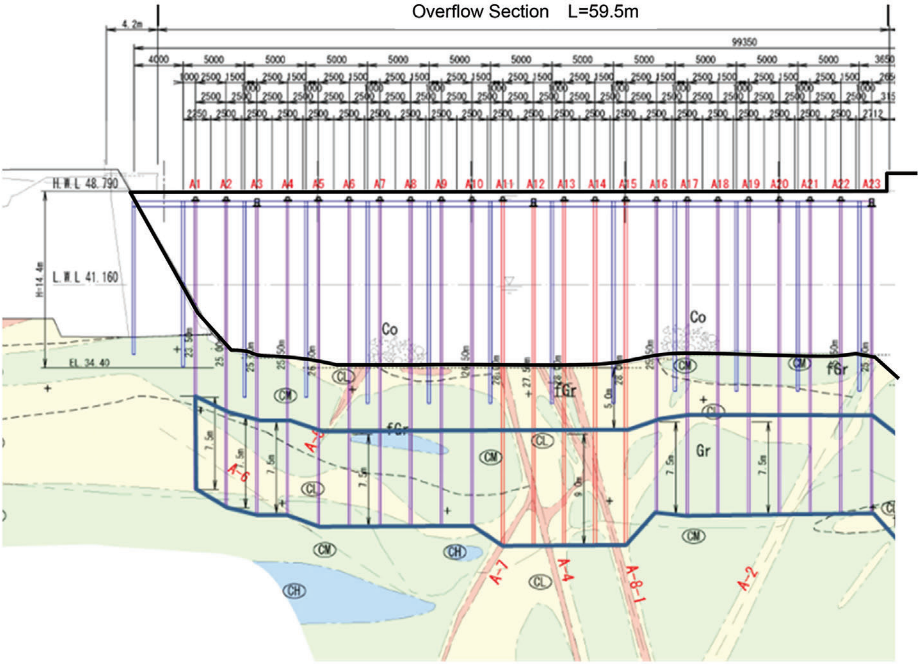
It was confirmed by two-dimensional FEM analysis whether the anchor tension was appropriately transmitted to the base of the dam. In this analysis, the dam and rock masses were modeled as planar strain elements and the anchors were modeled as truss elements, and the vertical stress distribution occurring at the base of the dam in normal and seismic conditions was compared with the presence or absence of anchor tension. For the cross-sectional model, we chose a place where the CM rock and the CL rock of the overflow section are mutually stratified.

### 4.2 *Preparation of two-dimensional FEM model (overflow part)*

Figure 8 shows the model diagram, load conditions, and physical property values. The main conditions for the analysis were as follows:

1. The foundation rock was modeled as horizontally distributed CL rock (thickness 5 m) interposed in CM rock.
2. Regarding the water level, the reservoir is always at full water level, and the dam downstream water level is at the top end of the secondary dam. Hydrostatic pressure and uplift pressure were applied as constant loading.

Left side of upstream view: Overflow Section




Right side of upstream view: Non-overflow Section

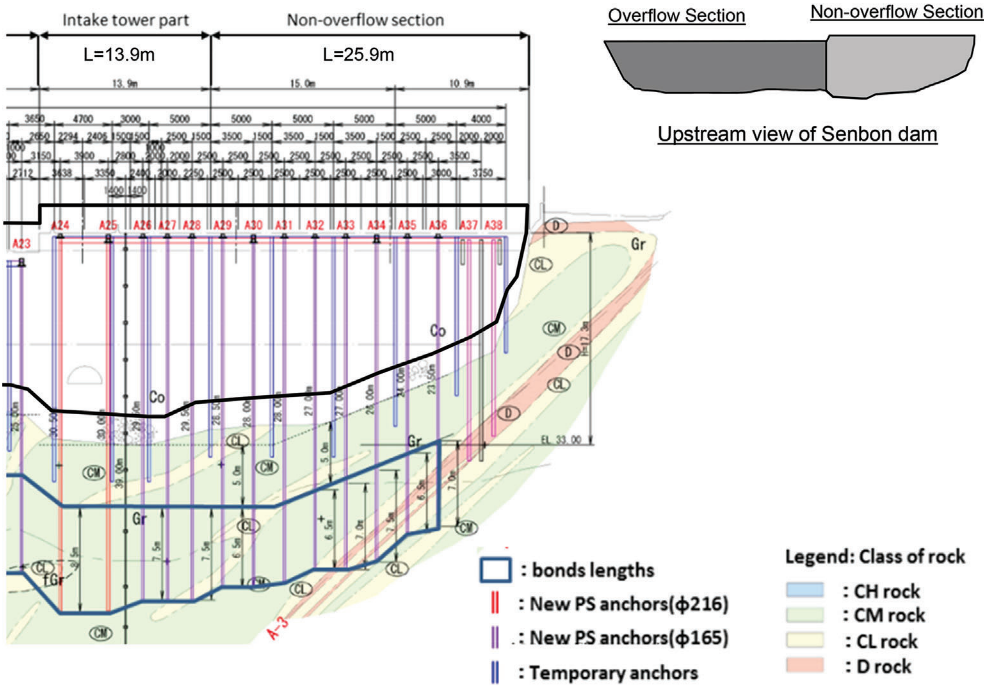



Figure 7. Longitudinal profile of Dam axis on layout of post tensioned anchors from upstream

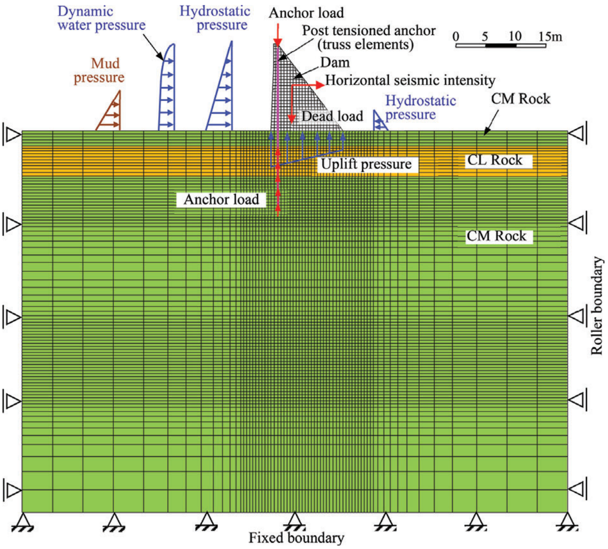



Figure 8. FEM model diagram and load condition

3. The earthquake load 0.12g was given in the horizontal direction as a static load. Also, hydrodynamic pressure was given to the upstream surface of the dam.
4. A design load of 1,099 kN was applied to the anchor head and the bond length portion.
5. In the bond length portion, the design load was divided into nodal loads every 1 m.

### 4.3 *Physical property values*

Physical property values for analyzing the dam and foundation were set based on the test values at the Senbon Dam and values of the Obara dam in the vicinity where the geology is similar. The physical property values used for the analysis are shown in Table 3.

### 4.4 *Results of two-dimensional FEM analysis*

The post-tensioned anchors improved the stress distribution at the base of the dam on the compression side, and the tensile stress in case of an earthquake changed to compressive stress. The results and considerations are described below (see Figure 9).

1. With the installation of post-tensioned anchors at the full water level (no earthquake), the stress distribution on the dam base surface increased to the compression side as a whole.
2. Tensile stress of up to 0.15 N/mm$^2$ was generated in the vertical direction at the upstream end of the dam at the time of an earthquake, but no tensile stress was generated after installation of the anchors, and the stress at the dam base changed to all compressive stress.

Table 3. Physical property values used for the analysis.

| | Dam | CM rock | CL rock |
|---|---|---|---|
| Uniaxail compression strength (N/mm$^2$) | 22.1 | 126.5 | 24.6 |
| Cohesion of shear strength (N/mm$^2$) | 0.82 | 1.52 | 0.66 |
| Friction angle (°) | 33 | 40 | 40 |
| Young‘s modulus (N/mm$^2$) | 18000 | 2000 | 600 |
| Poisson‘s ratio | 0.26 | 0.23 | 0.26 |
| Unit weight (kN/m$^3$) | 21.87 | 25.70 | 23.54 |

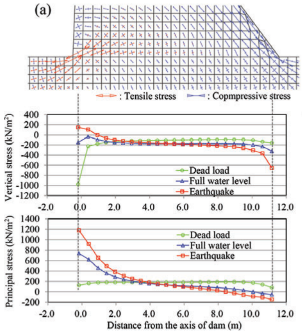



Figure 9. Comparison of the stress distribution on the dam base with and without post-tensioned anchor:

3. The maximum principal stress at the time of an earthquake was about 1.2 N/mm$^2$ tensile stress almost horizontally at the upstream end of the dam bottom, and decreased to 1.03 N/mm$^2$ after pc anchors. This tensile stress was caused by stress concentration effect due to the 90 degree corner at the dam-foundation interface, and it is considered these stresses are fictitious. Further, these tensile values are quite small compared to the average value of 22 N/mm$^2$ of the uniaxial compressive strength test of the Senbon Dam.

## 5 CONSERVATION OF DAM LANDSCAPE

The Senbon Dam is a gravity-type masonry dam completed in March 1918, just hundred years ago, and it is the 13th oldest among the 70 domestic masonry dams. Furthermore, the cultural property value is high from the unique masonry style and the beautiful shape of the overflow crest, and it is designated as a recommended civil engineering heritage in 2003 by the Japan Society of Civil Engineers and a national registered tangible cultural property in 2008. Therefore, it was strongly required not to change the dam appearance. The post-tensioned anchor method is an optimal choice to conserve heritage dams.

On the other hand, as shown in Figure 10, since the cross section of the dam body is slender and concrete strength is not high, it was also required to reduce the anchor load as much as possible. Therefore, measures such as the dispersion of load by narrowing anchor pitches and the reduction of the load in anchor heads by a new pre-tension method by transmitting the full anchor load to inside of the dam body are adopted. As for the overflow crest to be cut out for the anchor work, cutting areas will be decreased to a minimum by the above measures, and the masonry landscape will be restored with the stone which is close to the color of hundred years ago.

Figure 10. Photograph views of Senbon Dam: the sharp edge of the dam top creates hydraulic beauty.

## 6 CONCLUSIONS

Post-tensioned anchors have been used at many dams around the world, but the Senbon Dam is the first time example in 50 years in Japan. In the design, because the current domestic anchor standards are for soil anchors, we improved the design method for rock anchors proposed by the Japan Dam Engineering Center about 10 years ago with reference to overseas standards.

As a result, it was possible to carry out seismic reinforcement effectively in terms of construction cost and period, compared with conventional methods such as placing concrete to the upstream or downstream dam faces. It is confirmed by the two-dimensional analysis that the stress state of the dam body was clearly improved by installing post-tensioned anchors.

Actual site works of the Senbon Dam will be implemented from 2019 to 2020.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