Seismic Resistance Designs for the New Higashiyama Service Reservoir No.3

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Abstract
Nagoya Waterworks & Sewerage Bureau plans to implement renewal work on a decrepit service reservoir that was constructed at the Bureau's foundation. This paper introduces a practical case of seismic resistance designs for the RC rectangle service reservoir according to "Guideline to and Explanation of Seismic Construction Method of Water Supply Facilities-2009."

INTRODUCTION

Fig.1, Location of Nagoya

Photo 1, The Higashiyama Service Reservoirs

Nagoya city, the third largest city in Japan, is located in the center of Japan islands. Fig. 1 shows the location of Nagoya, where the "Nankai trough" extends in the south and is concerned as a possible cause of a major earthquake in the near future. Waterworks of Nagoya started in 1914. Its Higashiyama service reservoirs, consisting of 5 reservoirs (shown in photo 1), were constructed between 1914 and 1934 and they are the oldest service reservoirs in Nagoya.

The Higashiyama service reservoirs have been distributing tap water to the center of Nagoya city by natural gravity for over 100

Table 1, Specific of the Higashiyama Service Reservoirs

No.	Constructed Year	Improved Year	Shape	Structure	Improving Method	Capacity(m^3)	Planned Capacity(m^3)
1	1913	1999	Rectangle	Plain Concrete	Interior Strengthening	6,860	6,860
2	1913	2013	Rectangle	RC	Renewal	23,500	23,500
3	1928	-	Rectangle	RC	Renewal	9,370	23,500
4	1928	-	Rectangle	RC		9,370	
5	1934	2001	Rectangle	RC	Interior Strengthening	27,000	27,000
					total	76,100	80,860

years. The specifics of the 5 service reservoirs are shown in Table 1.
Although seismic strengthening work on the interior of reservoirs No.1 & No.5 was implemented because of their seismic vulnerability, total-capacity enlargement work on the Higashiyama reservoirs has also been required for saving distribution energy since 2010. Due to this requirement, as shown in a red rectangle of Table 1, reservoirs No.3 & No.4 were chosen to be renewed as "new service reservoir No.3" together with seismic resistance designs.

SEISMIC RESISTANCE DESIGNS

According to "Guideline to and Explanation of Seismic Construction Method of Water Supply Facilities-2009" (hereafter "JWWA Guideline 2009"), the seismic resistance designs for the new service reservoir No.3 proceeded as shown in Fig. 2.

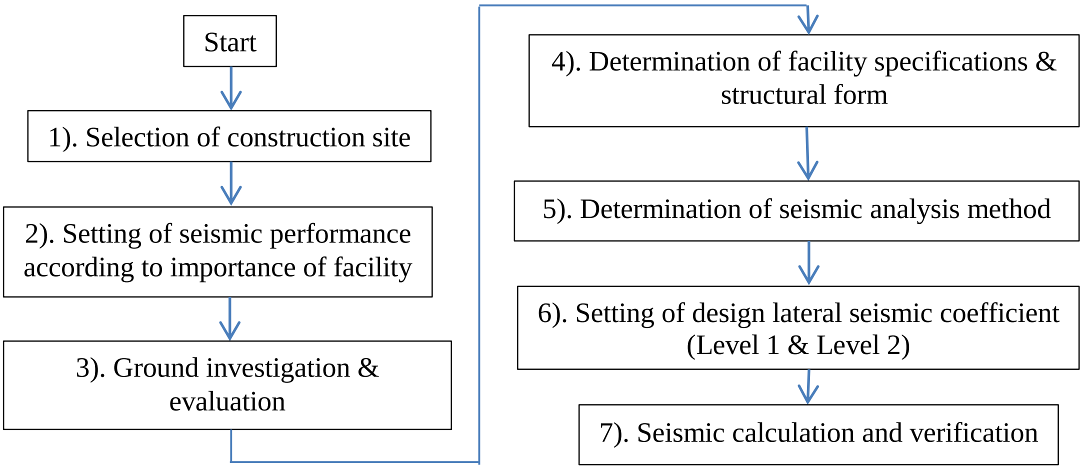


Fig. 2, Procedure of Seismic Resistance Designs

Selection of construction site

The construction site was limited to the existing service reservoirs No.3 & No.4, because of their altitude and facility capacity.

Table 2, Importance classification of service reservoir

Rank	Subject
A1	Having high probability of incurring serious secondary disaster if damaged
	Connecting to distribution main or maximum capacity and having no alternative facilities
A2	Connecting to distribution main and having alternative facilities
B	Other than those above

Table 3, Definition of seismic motion

Level 1	High occurring probability during operational period of subject facility
Level 2	Maximum intencity expected in the location of subject facility

Setting of seismic performance according to importance of facility

If damaged by earthquake, service reservoir No.3 may cause a major secondary disaster due to the collapse of the slopes from leakage. Thus, service reservoir No.3 is classified as "Rank A1." Table 2 shows classification of service reservoirs by importance according to JWWA

Guideline 2009.

Rank A1 facilities require “seismic performance 1” against “seismic motion level 1,” and “seismic performance 2” against “seismic motion level 2.” Table 3 shows the definitions of seismic motion level 1 and 2, and Fig. 3 explains seismic performance 1, 2, and 3 of RC service reservoir constructions. As shown in Fig.3, seismic performance 1 requires no leakage when small cracks occur. On the other hand, performance 2 allows small leakages which can be repaired within a short term.

Furthermore, Fig. 4 shows the concept image on the relation of response curvatures against bending moments for a part of a RC reservoir construction. Performance level 1 (Fig. 3, on the left) meets damage level 1 (elastic range), whereas performance level 2 (Fig. 3, in the middle) meets damage level 2 (in plastic range, less than max load bearing capacity).

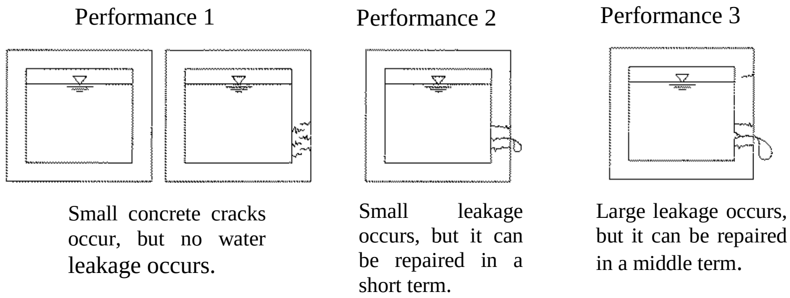


Fig. 3, Seismic Performance of RC Service Reservoir

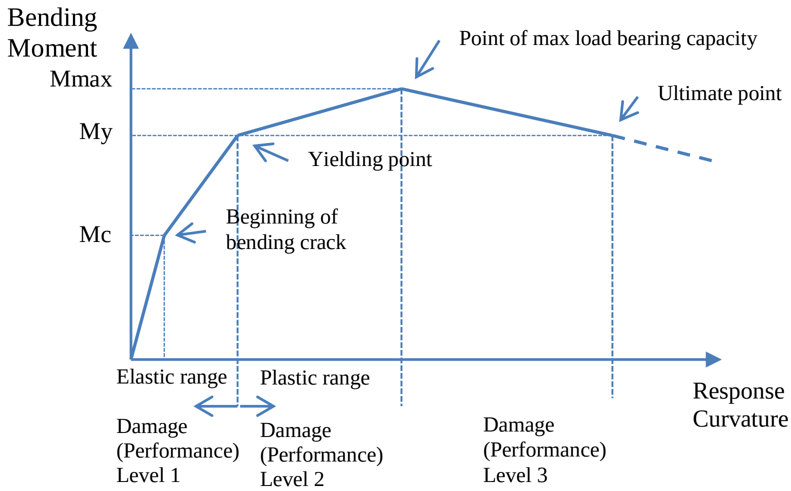


Fig. 4, Damage Level Against Bending Stress for a Part of Consruction

Ground investigation and evaluation

(1) Liquefaction

Based on a boring survey, it was determined that there were no sand layers which may cause liquefaction at the construction site.

(2) Surface of Engineering bedrock

The surface of engineering bedrock is an

Table 4, specific of basement of construction site

No. of layer	type of soil	H_i (m)	N value	V_S (m/s)	$4H_i/V_{Si}$ (s)	note
1	sandy	1.40	5	144.65	0.04	
2	sandy	2.90	35	319.71	0.04	
3	sandy	3.60	19	296.21	0.05	
4	sandy	1.00	11	276.65	0.01	
5	sandy	0.60	11	276.65	0.01	
6	sandy	2.20	10	273.37	0.03	
7	sandy	3.10	35	319.71	0.04	
8	cohesive	7.70	16	285.68	0.11	
9	cohesive	1.30	38	334.68	0.02	
10	sandy	0.90	54	337.52	0.01	
11	cohesive	2.30	35	329.68	0.03	
12	cohesive	1.90	18	291.91	0.03	
13	cohesive	0.50	22	302.83	0.01	
14	cohesive	1.60	26	312.23	0.02	
15	sandy	-	125	374.86	-	engineering bedrock
				T_G (s)	**0.43**	

upper surface of solid bedrock having enough large shear wave velocity compared with ground level, and it is used as reference bedrock for setting seismic motions. Usually, seismic waveforms used for seismic resistance calculation are ground level waveforms calculated with waveforms on the engineering bedrock. We set a series of bedrocks having more than 300m/s of shear wave velocity and more than 50 of N-value as engineering bedrock on this design.

(3) Basement Classification

Basement classification is used to calculate design lateral seismic coefficients or design displacement amplitude. As described hereafter, a static analysis method was adopted this time, which requires basement classification for calculating design lateral seismic coefficients. Basement classification is determined by the natural period of a basement "$T_{G.}$" T_G is calculated by the formula (1) below. Table 4 shows the specifics of basement at the construction site.

$$T_G = \mathbf{4\sum_{i=1}^{n} H_i/V_{Si}} \quad \cdot \cdot \cdot \cdot (1)$$

where,

T_G: Natural period of basement (s)

H_i: Thickness of No.i layer (m)

V_{Si}: Mean shear wave velocity of No.i layer (m/s)

i: Layer number from ground surface, when basement is divided from ground surface to engineering bedrock by n layers

As shown at the bottom of Table 4, the natural period of basement "T_G" was 0.43(s), resulting in type II of basement classification in Table 5.

Table 5, Basement Classification

Type	Natural Period
Ⅰ	$T_G < 0.2$
Ⅱ	$0.2 \leqq T_G < 0.6$
Ⅲ	$0.6 \leqq T_G$

Table 6, facility specifications & Structural form

capacity	23,500m^3
shapsize	49.000m×58.800m×4.550m×2tanks
structure	RC/flat slab
substructure	pile foundation(SC＋PHC)

Determination of facility specifications & structural form

Table 6 shows facility specifications and structural form of service reservoir No.3. Facility specification is determined by the planned capacity, low water level of other service reservoirs, conditions of basement,

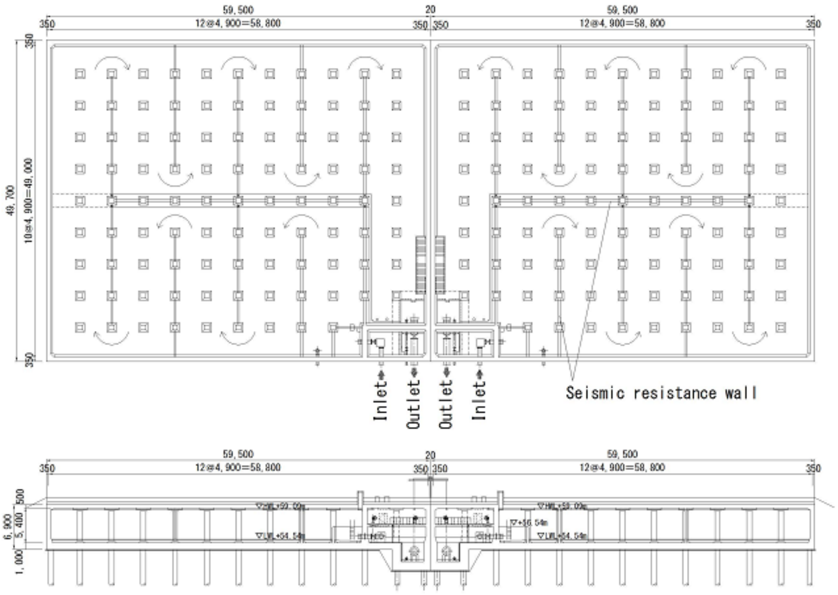


Fig. 5, Basic Structure

locations of existing pipes or facilities, and easy maintenance. Since RC rectangle service reservoirs have a structure with lower lateral stiffness than longitudinal stiffness against horizontal shaking in general, lateral shaking is the dominant influence. Thus, the effective seismic design is to allocate longitudinal & lateral seismic resistance walls in a good balance according to the shape of the service reservoir. From this reason, 5 lateral and 1 longitudinal seismic resistance walls were set for each tank as shown in Fig. 5.

Determination of seismic analysis method

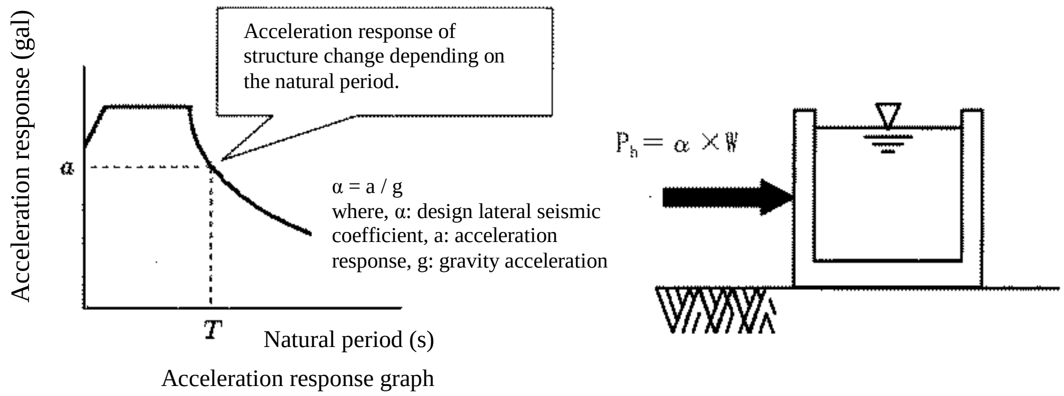


Fig. 6, Concept Image of Seismic Intensity Method

Service reservoir No.3 is an aboveground structure with soil cover, resulting in inertial force as the dominant influence during an earthquake. Therefore, we decided to apply a static analysis method (seismic intensity method). In the seismic intensity method, momentary force applied when the structure is displaced at maximum by earthquake is used as static horizontal force. Seismic force applied to the structure (= horizontal force “P_h”) is calculated by multiplying a design lateral seismic coefficient “α” with the structure weight “W.” A concept image of seismic intensity method is depicted in Fig. 6. Two-dimensional non-linearity frame,

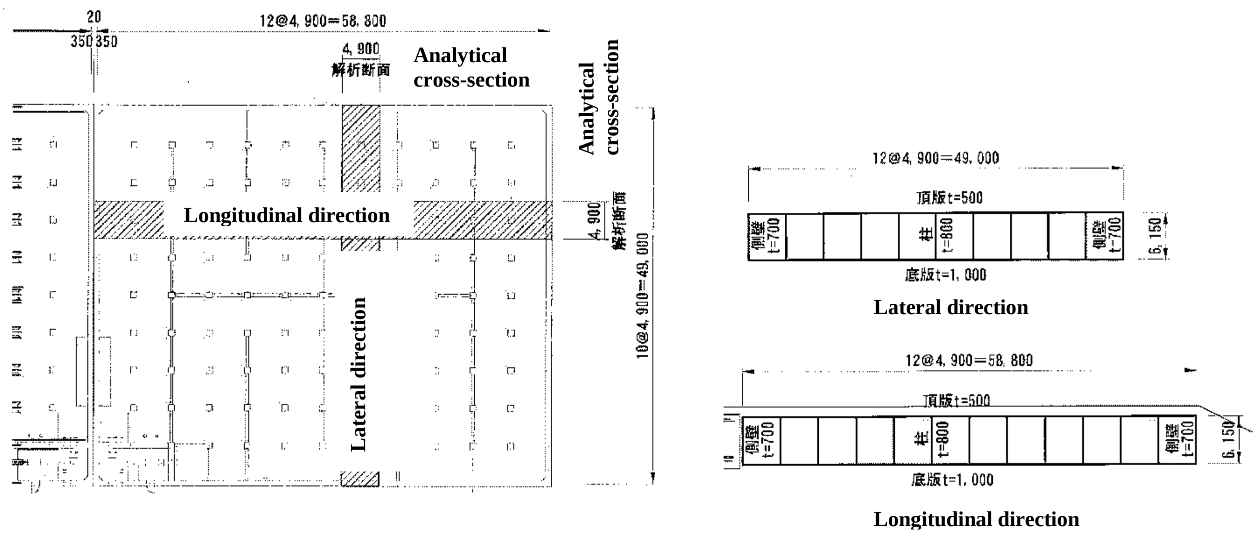


Fig. 7, Two-Dimensional Frame Model

used as analytical model, is shown in Fig. 7.

Setting of design lateral seismic coefficient

(1) Seismic Motion Level 1

According to JWWA Guideline 2009, a design lateral seismic coefficient of seismic motion level 1 “K_{h1}” is calculated by the following formula (2).

$K_{h1} = C_z \cdot K_{h01}$ ・・・・(2)

where,

K_{h1}: Design lateral seismic coefficient,

C_z: Area compensation coefficient (= 1.0 for No.3 construction site),

K_{h01}: Standard lateral seismic coefficient at gravity center of applicable structure

Table 7 shows the standard lateral seismic coefficients of seismic motion level 1 with the natural period of the structure “T,” according to each basement classification. As mentioned above, the type of basement at the construction site is classified as type II. Based on eigenvalue analysis, the natural period of structure was calculated as 0.04(s) for lateral and 0.05(s) for longitudinal direction. Therefore, the design lateral seismic coefficient “K_{h1}” is calculated as below.

Table 7, Standard lateral seismic coefficient for aboveground structure (Level 1)

Basement classification	Kh01(corresponding to natural period T)		
Type Ⅰ [TG＜0.2]	$T<0.10$ $K_{h01}=0.431T^{1/3}$ $K_{h01}\geqq 0.16$	$0.10\leqq T\leqq 1.1$ $K_{h01}=0.20$	$1.1<T$ $K_{h01}=0.213T^{-2/3}$
Type Ⅱ [0.2≦TG＜0.6]	$T<0.20$ $K_{h01}=0.427T^{1/3}$ $K_{h01}\geqq 0.20$	$0.20\leqq T\leqq 1.3$ $K_{h01}=0.25$	$1.3<T$ $K_{h01}=0.298T^{-2/3}$
Type Ⅲ [0.6≦TG]	$T<0.34$ $K_{h01}=0.430T^{1/3}$ $K_{h01}\geqq 0.24$	$0.34\leqq T\leqq 1.5$ $K_{h01}=0.30$	$1.5<T$ $K_{h01}=0.393T^{-2/3}$

Tg: Natural period of basement (s)

- Lateral direction: $K_{h01} = 0.427 \times 0.04^{1/3} = 0.146 \rightarrow 0.20$, $K_{h1} = 1.0 \times 0.20 = 0.20$
- Longitudinal direction: $K_{h01} = 0.427 \times 0.05^{1/3} = 0.157 \rightarrow 0.20$, $K_{h2} = 1.0 \times 0.20 = 0.20$

(2) Seismic Motion Level 2

Based on JWWA Guideline 2009, a design lateral seismic coefficient of seismic motion level 2 was selected from the largest of the intensities calculated by two methods; (a) “calculation method from seismic ground level motions anticipated by the regional disaster prevention plan in Nagoya” and (b) “calculation method based on the observation record of the Great Hanshin-Awaji Earthquake.” In the former method, two waveforms were

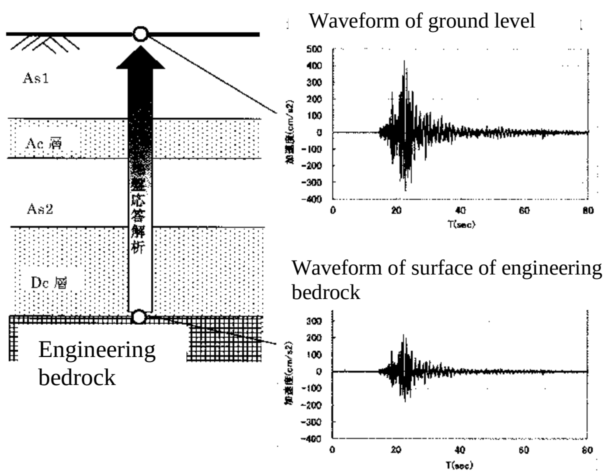


Fig. 8, Concept Image of ODERA

selected for “Nankai trough (see Fig. 1); the largest-class seismic ground level motion considered from the earthquakes in the past and the largest-class seismic ground level motion considered from every possibility.

(a) Design seismic lateral coefficient based on regional disaster prevention plan in Nagoya

Seismic ground level motion is amplified or damped while transiting trough subsurface ground. The maximum response acceleration can be determined by the one-dimensional earthquake response analysis (hereafter “ODERA”). Fig. 8 shows the concept image of this. In this design, ODERA was carried out to compute a ground level motion from the seismic motion of engineering bedrock of the “Nankai trough (see Fig. 1)” earthquake. Furthermore, because this analysis method has a variation in number of applications, convergence, and influence of frequency domain; three analysis methods were applied; “SHAKE*1,” “FDEL*2,” and “DYNEQ*3.” Table 8 shows maximum response accelerations computed by ODERA.

Table 8, result of ODERA

	Analysis code	Previouse earthquakes(gal)	Every possibility(gal)
Lateral (T=0.04)	SHAKE	296	319
	FDEL	326	373
	DYNEQ	317	352
Longitudinal (T=0.05)	SHAKE	305	328
	FDEL	351	421
	DYNEQ	351	381

As a result, the FDEL of every possibility showed the largest in both longitudinal & lateral directions, the design lateral seismic coefficient K_{h2} is calculated as below.

- Lateral direction: $K_{h2} = 373 / 980 = 0.38$
- Longitudinal direction: $K_{h2} = 421 / 980 = 0.43$

*1, University of California, Berkeley, USA, “A Computer program for earthquake response analysis of horizontally layered sites”

*2, Gifu University, Japan, a computer program based on “Frequency-Dependent Equivalent Strain for Equi-Linearized Technique”

*3, Tohoku Gakuin University, Japan, “A computer program for dynamic response analysis of level ground by equivalent linear method”

(b) Design lateral seismic coefficient based on the observation record of the Great Hanshin-Awaji Earthquake

Table 9 shows standard lateral seismic coefficients of Level 2 for aboveground structures according to JWWA Guideline 2009. These were developed based on the observation record of the Great Hanshin-Awaji Earthquake, which caused major damage to many structures. As mentioned above, the basement classification is type II, and the natural periods of structure T are 0.04(s) in the lateral, 0.05(s) in the longitudinal direction. Thus, a design

Table 9, Standard lateral seismic coefficient for abovegound structure (Level 2)

Basement classification	Kh02(corresponding to natural period T)		
Type Ⅰ [TG＜0.2]	T＜0.20 $K_{h02}=2.291T^{0.515}$ $K_{h02}\geqq 0.70$	0.20≦T≦1.0 $K_{h02}=1.0$	1.0＜T $K_{h02}=1.000T^{-1.465}$
Type Ⅱ [0.2≦TG＜0.6]	T＜0.20 $K_{h02}=5.130T^{0.807}$ $K_{h02}\geqq 0.80$	0.20≦T≦1.0 $K_{h02}=1.4$	1.0＜T $K_{h02}=1.400T^{-1.402}$
Type Ⅲ [0.6≦TG]	T＜0.30 $K_{h02}=2.565T^{0.631}$ $K_{h02}\geqq 0.60$	0.30≦T≦1.5 $K_{h02}=1.2$	1.5＜T $K_{h02}=2.003T^{-1.263}$

Tg: Natural period of basement (s)

lateral seismic coefficient K_{h2} is calculated as below.

- Lateral direction: $K_{h02} = 5.130 \times 0.04^{0.807} = 0.382 \rightarrow 0.80$, $K_{h2} = 1.0 \times 0.80 = 0.80$
- Longitudinal direction: $K_{h02} = 5.130 \times 0.05^{0.807} = 0.457 \rightarrow 0.80$, $K_{h2} = 1.0 \times 0.80 = 0.80$

By comparing the results of the two abovementioned methods, a design lateral seismic coefficient of seismic motion level 2 was determined as 0.80 in both directions.

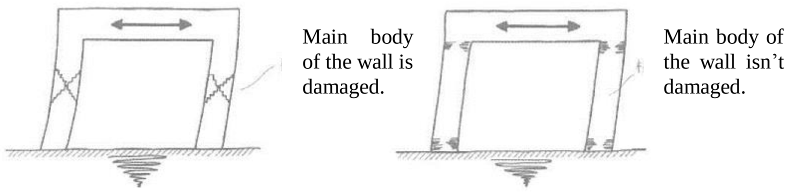
Seismic calculation and verification

Firstly, prior to seismic calculation, the maximum bending moments under normal conditions were determined using the two-dimensional frame model (see Fig. 7) in order to verify that occurring degree of bending stress on the respective parts is below the allowable degree. Secondly, seismic calculation was carried out using lateral seismic coefficients of seismic motion level 1 and level 2, and then the maximum bending moments and maximum shear forces on the respective parts were calculated. Finally, it is verified if occurring degrees of bending stress or shear forces calculated satisfies the following 5 requirements. If any of the requirements wasn't satisfied, thickness of wall or installation of reinforced steel was changed until all the requirements were satisfied. Table 10 shows the seismic calculation results. The first row indicates analytical cross sections, and the second row shows the respective applicable parts. The first column from the left shows analytical conditions and judgement criteria, and the second column shows items.

Requirements

- ✓ Occurring degree of bending compressive stress by seismic motion level 1 is below the allowable degree of bending compressive stress (see the seventh and eighth row in Table 10).
- ✓ Occurring degree of bending stretching stress by seismic motion level 1 is below the allowable degree of bending stretching stress (see the ninth and tenth row).
- ✓ Occurring bending moment by seismic motion level 2 is below the max load bearing capacity (see the eleventh and twelfth row).
- ✓ Occurring shear force by seismic motion level 2 is below the shear capacity (see the thirteenth and fourteenth row).
- ✓ Bending fracture precedes shearing fracture by seismic motion level 2 (see the bottom row).

To avoid dangerous momentary collapse, it is desirable that the occurring bending fracture is earlier than shearing fracture, and the structure is made tenacious. Fig. 9 shows the concept image about



Dangerous shear fracture   Tenacious Bending fracture

Fig. 9, Concept image of fracture mode

this. The verifying formula is as below.

$\gamma_i \cdot V_{mu}/V_{yd} < 1.0$

where,

γ_i: Structure coefficient (= 1.0),

V_{mu}: Shear force when the applicable part reached the max load bearing (kN),

V_{yd}: Shear capacity (kN)

Table 10, Results of seismic calculation

Analytical cross-section			Lateral direction						Longitudinal direction					
Judgement	Item	Part / unit	Upper slab	Side wall (downside)	Side wall (upside)	Column	Bottom slab (upside reinforcing steel)	Bottom slab (downside reinforcing steel)	Upper slab	Side wall (downside)	Side wall (upside)	Column	Bottom slab (upside reinforcing steel)	Bottom slab (downside reinforcing steel)
Normal (degree of stress ≦ allowable degree of stress)	Degree of bending compressive stress σ_c	N/mm^2	5.454	2.926	4.362	4.260	2.330	2.957	5.268	6.463	4.253	6.864	3.250	2.938
	Allowable degree of bending compressive stress σ_{ca}	N/mm^2	9.000	9.000	9.000	9.000	9.000	9.000	9.000	9.000	9.000	9.000	9.000	9.000
	Degree of bending stretch stress σ_s	N/mm^2	155.208	82.184	139.438	17.256	143.359	168.107	168.986	165.248	157.903	67.652	175.864	140.947
	Allowable degree of bending stretch stress σ_{sa}	N/mm^2	180.000	180.000	180.000	180.000	180.000	180.000	180.000	180.000	180.000	180.000	180.000	180.000
Level 1 (degree of stress ≦ allowable degree of stress)	Degree of bending compressive stress σ_c	N/mm^2	8.554	12.051	7.122	12.560	5.675	5.231	8.838	12.335	7.374	13.299	5.846	5.264
	Allowable degree of bending compressive stress σ_{ca}	N/mm^2	13.500	13.500	13.500	13.500	13.500	13.500	13.500	13.500	13.500	13.500	13.500	13.500
	Degree of bending stretch stress σ_s	N/mm^2	243.434	286.521	242.521	253.705	282.615	250.936	251.529	293.801	251.947	246.866	289.788	252.527
	Allowable degree of bending stretch stress σ_{sa}	N/mm^2	300.000	300.000	300.000	300.000	300.000	300.000	300.000	300.000	300.000	300.000	300.000	300.000
Level 2 (section force ≦ load bearing avility)	Bending moment M_d	KN・m	344.070	5797.975	2589.559	2406.938	4101.169	3991.563	361.350	5975.411	2727.907	2533.940	4271.427	4014.976
	Max load bearing capacity M_{ud}	KN・m	370.756	6460.747	3585.079	2575.248	4466.711	4083.872	430.775	6456.424	3574.343	2682.453	4501.010	4083.872
	Shear force V_d	kN	237.270	2528.451	1389.860	1506.488	1721.425	1721.425	245.320	2535.235	1481.556	1491.734	1741.932	1741.932
	Shear capacity V_{yd}	kN	355.725	2573.399	2341.170	1763.796	3111.185	2881.004	365.743	2576.919	2345.828	1765.184	3121.630	2881.004
Judgement of flacture mode ($\gamma i \cdot Vmu/Vyd<1.0$ ⇒OK)	Bending moment M_d	KN・m	344.07	5797.98	2589.56	3158.12	4669.78	3991.56	361.35	5975.41	2727.91	3277.75	4847.21	4014.98
	Max load bearing capacity M_{ud}	KN・m	440.63	4785.04	2664.33	2114.03	5399.09	4882.79	510.62	4783.03	2657.29	2111.68	5433.60	4882.79
	Shear force V_d	kN	237.27	3066.01	1389.86	1506.49	1721.43	1721.43	245.32	3070.74	1481.56	1491.73	1741.93	1741.93
	Shear capacity V_{yd}	kN	479.92	3437.35	3105.13	2277.35	4165.00	3850.77	494.14	3440.68	3111.74	2278.99	4177.97	3850.77
	Shear span $L=M_d/V_d$	m	1.45	1.89	1.86	2.10	2.71	2.32	1.47	1.95	1.84	2.20	2.78	2.30
	$V_{mu}=M_{ud}/L$	kN	303.90	2531.80	1432.40	1006.70	1992.30	2104.60	347.40	2452.80	1444.20	959.90	1954.50	2123.00
	$\gamma_i \cdot V_{mu}/V_{yd}$		0.63	0.74	0.46	0.44	0.48	0.55	0.70	0.71	0.46	0.42	0.47	0.55

SUMMARY

This paper explained an example of seismic resistance designs according to the JWWA Guidelines 2009. As design lateral seismic coefficients are the key factors, we implemented ODERA with three different methods, based on seismic motions of engineering bedrock anticipated for the “Nankai trough (see Fig. 1)” earthquake from the regional disaster prevention plan in Nagoya, in order to determine the design lateral seismic coefficient. However, the design lateral seismic coefficient based on the “observation record of the Great Hanshin-Awaji Earthquake” surpassed it, and was adopted for the data in seismic motion level 2. Now we are preparing the order of renewal for service reservoir No.3. It would be a great pleasure, if this paper would be of help for some other waterworks design.

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