A STUDY ON ASEISMIC VERIFICATION AND RETROFIT METHODS FOR AN ELEVATED WATER TANK AGAINST STRONG EARTHQUAKES

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SUMMARY

This paper reports a case study on an aseismic verification procedure and seismic retrofit for an existing elevated steel water tank. As nature of elevated steel water tanks during earthquakes, such as the three-dimensional shape, center of mass located in high position and dynamic interaction between structure and contained water, complicated dynamic behavior of structures is expected. Because of both complicated structural behavior and large design earthquake (level 2 seismic motion), seismic diagnosis and seismic retrofit for the existing tanks have become a remarkable issue to be solved. The authors propose an aseismic verification procedure in this paper. Also retrofit techniques such as strength increase and seismic isolation, which are possibly effective to existing elevated tanks, are discussed.

In the proposed procedure, a finite element analytical approach, which can deal with an interaction between structural elements and liquid elements, is adopted to verify the seismic stability of the existing elevated steel water tank as a case study. It was confirmed that the procedure based on the use of a finite element analytical technique was effective and much rational compared with the conventional static approach with regard to analytical treatment of the behavior of structure-liquid interaction during earthquakes. Also from a case study result of seismic retrofit for the existing elevated steel water tank, it was confirmed that a relatively ready seismic retrofit method is very effective to keep the tank functional and after large earthquakes. The applied procedure and the used retrofit method can be useful for similar practical issues.

INTRODUCTION

The Seismic Design Guideline for Waterworks 1997[9] prepared by the Japan Water Works Association (hereinafter referred to as JWWA Guideline), have been revised after 1995 Hyogo-ken Nanbu earthquake. JWWA Guideline now considers two levels of seismic motion, so-called level 1 and 2 of seismic motion. Level 1 seismic motion corresponds to the conventional design seismic force assumed in the seismic coefficient method. On the other hand, level 2 seismic motion which is required in the Guideline as well as level 1 seismic motion is almost the same as 1995 Hyogo-ken Nanbu Earthquake. In adopting the design method assuming two levels of seismic motion, seismic performance demand of water supply facilities was defined in terms of the seismic motion level and the importance of the facilities as shown in Table 1. The Guideline stipulates that "when the structure is large and of a particular importance, and has a complex shape, dynamic analyses should be made whenever required in view of the characteristics of the structure, to verify its safety" in conjunction with the conventional seismic coefficient method.

The structure to be studied as shown in Figure 1 is ranked A in importance under the classification of the

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Guideline. The body of the tank is a steel shell, which is supported by a steel shell column at the center and 20 external steel pipe legs placed in a circle, via a ring beam in the tank. Installed within the tank are piers, ceiling supports and hangers of side panels to the tank. The base of each leg is connected to a reinforced concrete underground beam via each anchorage, which is supported by reinforced concrete piles with a 33-m length.

Table 1: Levels of seismic performance demanded of water supply facilities[9]

Importance	Level 1 seismic motion	Level 2 seismic motion
Rank A	No damage should be obtained.	No serious damage to human lives should be incurred. The facilities should remain functional even with minor damage to individual facilities.
Rank B	Facilities should remain functional even with minor damage to individual facilities.	The water supply system as a whole should remain functional even with structural damage to individual facilities. Early restoration should be possible.

For level 2 seismic motion, evaluation of seismic performance by generally applied static analyses was extremely difficult because the structure was of great importance, its three-dimensional structure caused complicated behavior during an earthquake. Also because the coupled behavior of the flexible steel shell tank and the water inside was expected to influence overall behavior of the structure. Then dynamic analyses were adopted in order to take into consideration the above-mentioned feature of the structure. The Guideline, however, provides no specific methods either for establishing conditions of dynamic analyses or for verifying analytical results. Because of no cases of application of the Guideline in practice since it has been revised, it was, therefore, necessary to propose a seismic verification procedure based on appropriate analytical and verification methods. This paper describes the results of a seismic diagnosis and retrofit for the existing elevated steel water tank based on the definition of the seismic performance and the seismic limit state for each element applied in this study.

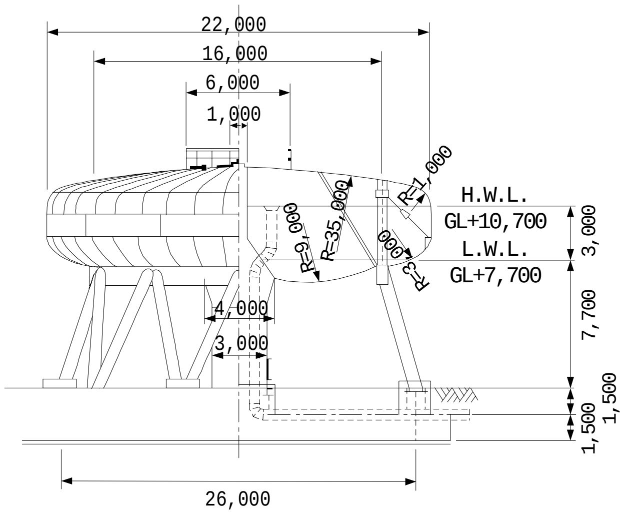


Figure 1: Elevated steel water tank to be reviewed

PRINCIPLES OF SEISMIC DIAGNOSIS AND SEISMIC VERIFICATION

As described above, verification for level 2 seismic motion was conducted using response obtained by dynamic analysis. In order to meet the requirements of seismic performance to a ranked A structure due to JWWA Guideline as listed Table 1, members of the structure were classified from a viewpoint of structural stability of the entire system and post-earthquake function[1],[8]. Limit states listed in Table 2 for individual members were defined for the members as bases for checking. The corresponding verification criteria were established as follows;

Table 2: Limit states for each element

Element	Limit state	Reasons for selection
Outer shell of tank	Serviceability limit state	Damage to the element may cause instability of the entire structure, or disrupt post-earthquake operation.
Internal cylinder		
Center column		
External leg		
Lower ring beam		
Upper ring beam	Ultimate limit state	Damage to the element has no influence on the stability of the entire structure or does not disrupt post-earthquake operation.
Piers within the tank		
Hanger		
Ceiling support		
Under ground beam		
Pile foundation		

The steel legs and the lower ring beam of the structure (Figure 2) were so important to structural stability that no damage affecting stability was allowable. The serviceability limit state was, therefore, accepted as a limit state for these members. Similarly, for the outer shell of the tank (except the ceiling), the serviceability limit state was adopted because the member was important, having a direct influence upon the post-earthquake function of the facility. The serviceability limit state for these steel members was defined as a state when analytically obtained stress reaches a yield stress level. This was because it was considered necessary for major steel members to resist forces within their elastic range of stresses as post-yield behavior of steel structures was not always clear and strength reduction could not be represented properly in analyses. For the other members, the ultimate limit state was considered because minor damage was not expected to cause instability of the entire structural system and it was also possible to keep the structure functional. Thus severe damage like yielding and fracture was accepted in such steel members as the ceiling and the hanger. The limit state for the foundation in this study is basically set slightly higher than the ultimate limit because heavy damage to foundations led to no collapse of the entire superstructure and consequently foundations were used without retrofit after an earthquake in some cases as seen in a report[2] on case studies of 1995 Hyogo-ken Nanbu Earthquake.

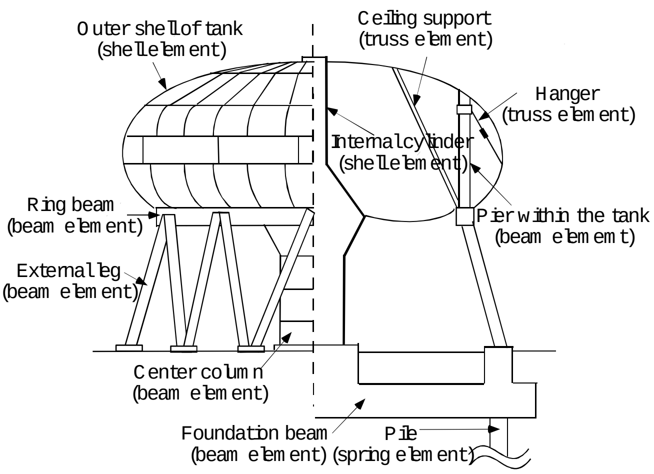


Figure 2: Members and modeling elements

ANALYTICAL METHOD AND BASIC CONDITIONS FOR ANALYSES

In modeling an elevated steel water tank, (i) coupled vibration of water tank and the liquid, (ii) damping properties of the liquid and (iii) evaluation of rigidity of steel legs were regarded as important factors. Concerning dynamic behavior of liquid containers, phenomena unique to a structure-liquid coupled system such as sloshing and building were well found. Various studies have been made so far[4],[13]. In view of the those studies, a finite element method, in which structures in a complicated shape can be modeled relatively precisely, was adopted in this study, and a three-dimensional model was used in order to analytically deal with the shape of the elevated steel water tank (Figure 3). The applied conditions for analyses are listed in Table 3. The liquid was assumed to behave in accordance with the linear velocity potential theory, and the interaction force conveyed at the boundary between the liquid and the tank body was represented as normal hydrodynamic pressure against the water tank element[11]. The water tank body was modeled as shell elements while internal piers, ring beams and legs were modeled as beam elements. The supporting legs, which resist large inertia forces, were considered the key part in terms of seismic performance of the structure. The elements were treated as elastic elements.

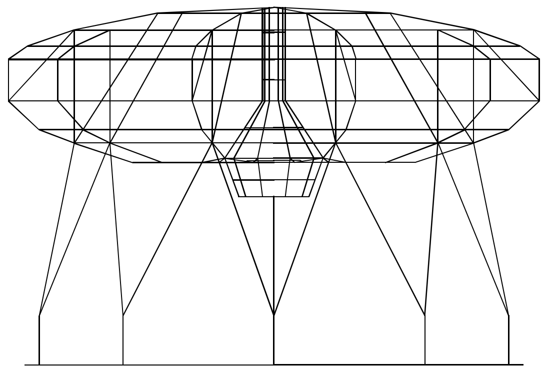
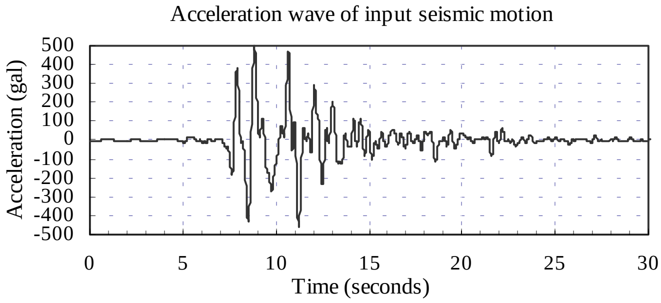


Figure 3: Model for analysis

Table 3: Conditions for dynamic analyses

Analytical method	Direct integration method, linear analysis	
Numerical integration method	Newmark's β -method (β =0.25,integration time interval:0.01 second)	
Damping type	Element stiffness proportional damping	
Damping coefficient of material	Outer shell of tank, internal cylinder, hanger, rafter, ceiling support, upper and lower ring beams, pier in the tank	0.03
	Center column, external legs	0.05
	Soil spring	0.3
	Underground beam	0.1
	Liquid (water)	0.01
Input seismic motion	Acceleration, obtained by soil response analysis by SHAKE, is adjusted to a maximum of 500 gal.	

Element stiffness proportional damping was selected to form the entire damping system. Damping coefficients of each structural members were fixed based on the values given in the Specifications for Highway Bridges[9]. Details of analytical conditions set in this study are explained in the reference[5]. The seismic excitation for dynamic analysis was obtained by using an equivalent linear analyses program (SHAKE[10],[12]) for ground to be studied, based on the one-dimensional multiple reflection theory (Figure 4). Then the N-S component of the 1995 Hyogo-ken Nanbu Earthquake record collected at the Kobe Marine Observatory was used as the input wave at a seismic base ground level. The amplitude of response acceleration wave at the pile head of the foundation was adjusted to a maximum of 500 gal for use as an input seismic motion in analyses. TDAP III[3],[14], a general-purpose finite element analysis software product, was used for the analyses.



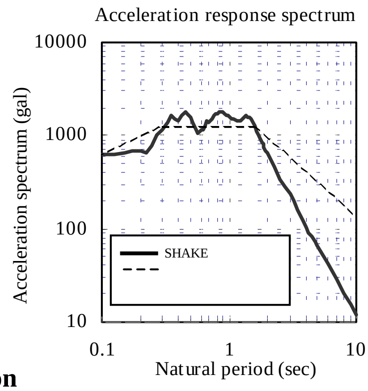


Figure 4: Input seismic motion

RESULTS OF DYNAMIC ANALYSES

The results of dynamic analyses are shown in Table 4 and Figure 5. As seen from time history response wave, the structure has response of a relatively short period of about 0.5 second. The natural period of the structure without the water is about 0.15 second, and the coupled effect of the structure and the water seems to make the period of response longer. According to the response spectrum of the input acceleration (Figure 4), the maximum acceleration response at a period of 0.5 second is approximately over twice that at a period of 0.15 second. The maximum response displacement is about 6 cm on the crest of the water tank so that the structure as a whole is relatively rigid and deformation due to bending vibration is unlikely to occur.

The analysis results show that little sectional force acts on external legs except axial forces. This means that the external legs behave as truss members. While an axial force of 70 tf acts under the self-weight, an axial force three to four times that level (310 tf) occurs due to seismic effects. On the center column, no changes in axial force are observed. A maximum moment of 2600 tfm occurs at the bottom edge of the center column. Thus it is clear that the center column behaves as a bending member.

Table 4: Maximum response sectional forces in supports

	Top of column	Bottom of column	Top of leg	Bottom of leg
Axial force	265.4tf	269tf	310tf	310.4tf
Shear force	411.6tf	411.2tf	0.1tf	0.7tf
Bending moment	705.2tfm	2640tfm	3.2tfm	0.0tfm

RESULTS OF SEISMIC PERFORMANCE CHECK

The center column and external legs, which were the key parts to keep stability of the structure, were checked for stress using the results of the dynamic analysis based on the principles described in 3 above. For members which are subject to both axial force and bending moment, stresses and stability generally need to be checked[7]. The verification results in terms of seismic performance are shown in Table 5. Seismic verification was made for the top and bottom ends of the center column and the top and bottom ends of external legs where the maximum sectional forces were observed during the dynamic analysis. The values less than 1.0 in the table indicate that the checking criterion for the element is satisfied. As seen from the table, the criteria are not satisfied in almost all elements and for nearly all items.

Judging from the results of stress checks, shear stress combined with bending stress on the top and bottom ends

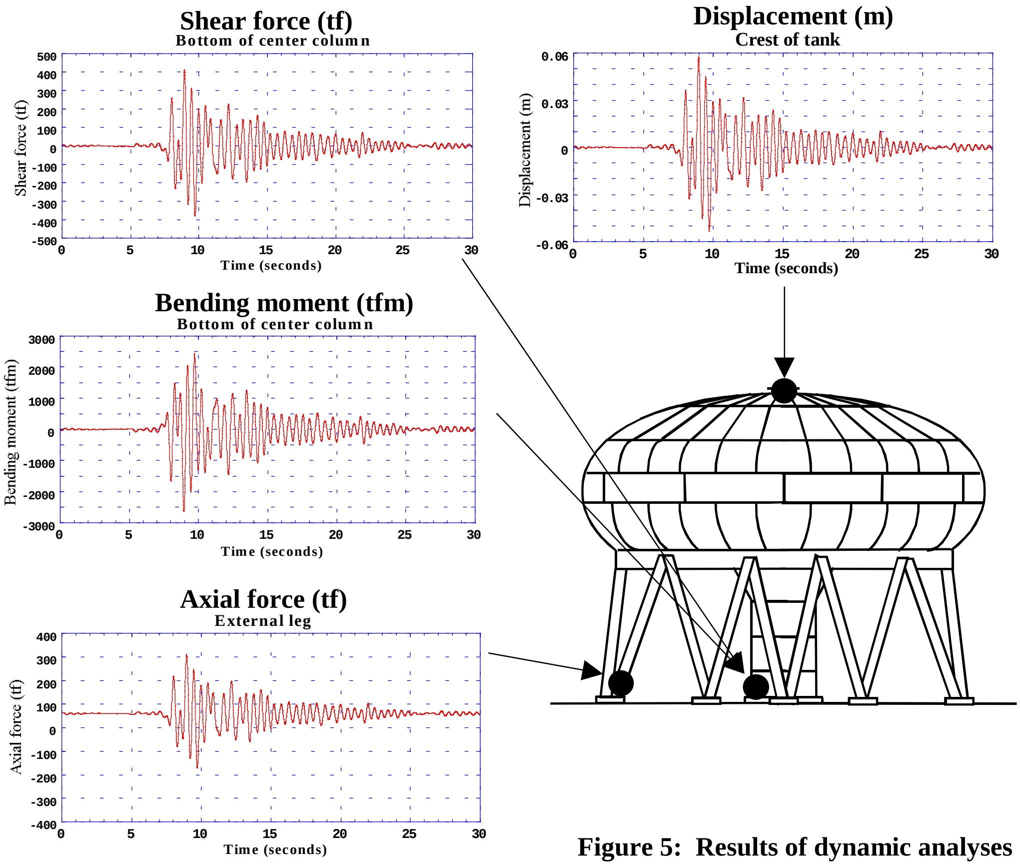


Figure 5: Results of dynamic analyses

Table 5: Verification results for supports

Verification item	Checking point			
	Top of column	Bottom of column	Top of leg	Bottom of leg
Compressive stress	1.03	3.19	1.75	1.80
Shear stress	2.07	2.07	**0.04**	**0.04**
Composite stress	12.67	12.61	1.95	1.99
Buckling stability	**0.76**	2.21	1.99	2.04

Less than 1.0:criterion satisfied, 1.0 or larger: criterion not satisfied

of the center column, and compressive stress due to bending moment at the bottom end far exceed the allowable values for the center column. For external legs, while compressive stress due to bending moment is much greater than the allowable value, shear stress is well below the allowable level. These results are considered attributable to large shear force and bending moment on the center column, and to the predominance of axial compressive force in external legs. It was found that the stability of the structure resulted in the possibility of buckling at all examined points, and thus the strength of members was not at the satisfactory level to keep the seismic performance of the structure.

SEISMIC PERFORMANCE IMPROVED BY RETROFIT

A seismic diagnosis of the present structure revealed the lack of strength of the center column and the external legs. Several retrofit techniques can be used for this case study. In order to improve seismic performance of the tank, strength increase technique and seismic isolation technique were examined as summarized in Table 6. Specific retrofit methods to increase the strength of the structure such as concrete in-filled and steel jacketing (steel sheet lining) were evaluated in terms of their advantages and disadvantages. Similarly, seismic isolation bearing system and tuned mass damper system to shift a peak vibration period of the structure towards a range of relatively small response of the input seismic motion and absorb energy of the input seismic motion were evaluated. From the evaluation in Table 6, it was concluded that concrete in-filled method and steel jacketing method in the strength increase technique were more superior than the seismic isolation technique.

Table 6: Retrofit methods considered in this study

Retrofit technique	Methods	Advantages	Disadvantages
Strength Increase	Concrete in-filled	▪ Low construction cost ▪ Easiness of retrofit works ▪ Aesthetic view	▪ Difficulty of perfect concrete in-filled ▪ Uncertainty of effectiveness due to the above reason
	Steel jacketing (steel sheet lining)	▪ Certainty of effectiveness ▪ Easiness and certainty of retrofit works ▪ Aesthetic view	▪ Difficulty of set of steel sheet lining at the both ends
Seismic Isolation	Seismic isolation bearing system	▪ Easiness for set of bearing system	▪ Unefficiency of seismic isolation for use of axial force reduction ▪ Satisfaction of required issues for normal time situation
	Tuned mass damper system	▪ Certain effectiveness if it works ▪ Easiness for set of TMD	▪ Limit of amount of mass ▪ No effectiveness unless additional mass is enough

Then in view of practical effectiveness, the past experience of retrofit and aesthetics, steel jacketing (steel sheet lining) was planned, and efforts were made to identify the specifications for retrofit to provide the required seismic performance, although little difficulty of set of steel sheet lining at the both ends of supports were expected. The minimum level of retrofit was, however, aimed at, which might lead to certain degrees of damage but cause no instability of the entire structure. These were based on the consideration that increasing the thickness of the steel sheet used for retrofit would not necessarily lead to an increase in cost-performance, and that large-scale earthquakes were assumed. Some trail analyses for a retrofit method using tuned mass damper system were carried out. It was found that effective result was obtained from the system with lager additional mass which the existing structure was not able to support the weight due to the tuned mass damper system without strengthening for the entire structure. It could be possible to strengthen the entire structure with costly construction fee and therefore it was determined that a cost-benefit balance was inferior.

In defining specifications for retrofit, methods based on dynamic analyses were used as in seismic diagnoses. Dynamic analyses and verification were conducted using the thickness of steel sheet, used for lining the center and external supports, as parameters, and appropriate specifications for retrofit were finally determined. As a result, it was found that lining of center and external supports with 9-mm-thick steel sheets could meet a requirement for seismic performance of the structure against the level 2 seismic motion.

The results of dynamic analyses are shown in Table 7 and Figure 6. With an increase of rigidity of the entire system by retrofit, the displacement on the crest decreases from the pre-strengthening level of 6 cm to 5 cm. Bending moment and axial force are predominant in the center column and external legs, respectively. Thus the vibration characteristics of the whole structure is almost the same as at present. Sectional forces acting on the center and external supports are, however, different. Bending moment and shear force occurring on the center column are approximately 40% smaller than at present. This can be because an increase in the strength of the external legs increases the rigidity of a truss structure consisting of external legs and the upper ring beam, and the motion of the water tank is constrained. As a result, sectional force acting on the center column is reduced. As the truss structure becomes stronger, deformation in supports decreases and overall swaying and rocking become predominant. Such behavior reduces the force on the center column.

Table 7: Maximum response sectional force of supports (after retrofit)

	Top of column	Bottom of column	Top of leg	Bottom of leg
Axial force(tf)	262.8 (265.4)	266.8 (269)	350.6 (310.0)	351.6 (310.4)
Shear force(tf)	249.0 (411.6)	247.8 (411.2)	2.0 (0.1)	2.8 (0.7)
Bending moment(tfm)	581.6 (705.2)	1749.4 (2640.0)	10.6 (3.2)	5.8 (0.0)

Note: Figures in the parentheses indicate the present values.

The results of verification of major elements of the structure based on the results of the dynamic analysis are shown in Tables 8 and 9. The results are within the allowable range for the center and external supports, which means that retrofit provides sufficient strength to the supports. Verification criteria are also satisfied for the

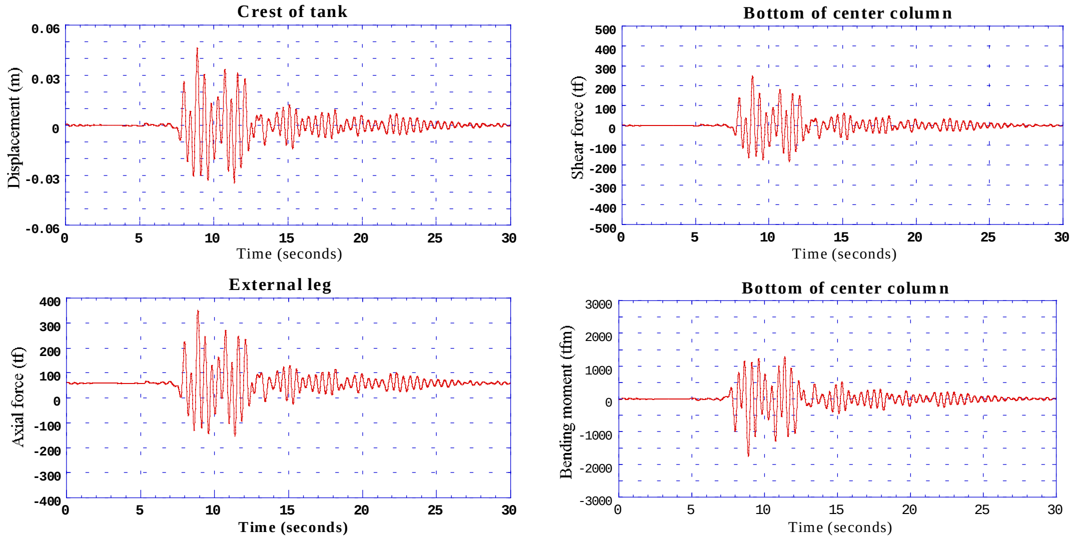


Figure 6: Results of dynamic analyses (after retrofit)

Table 8: Verification results for supports (after retrofit)

Verification item	Checking point			
	Top of column	Bottom of column	Top of leg	Bottom of leg
Compressive stress	0.32	0.80	0.85	0.78
Shear stress	0.31	0.30	0.02	0.03
Composite stress	0.32	0.80	0.95	0.88
Bucking stability	0.30	0.74	0.95	0.88

Less than 1.0:criterion satisfied, 1.0 or larger: criterion not satisfied

Table 9: Verification results for each element after retrofit

Element	Limit state	Verification results
Outer shell of tank	Serviceability limit state	Undergoes neither yielding nor buckling
Internal cylinder		
Center column		
External leg		
Lower ring beam		
Upper ring beam	Ultimate limit state	Undergoes neither yielding nor bucking
Pier within the tank		Undergoes partial yielding nor bucking
Hanger		Undergoes total bucking
Ceiling support		Undergoes total bucking
Underground beam		Undergoes partial damage due to bending
Pile foundation		Undergoes damage due to shear at the pile head

lower ring beam and the outer shell of the water tank. The structural elements, which are critical to post-earthquake stability and function of the structure, are found to have sufficient earthquake resistance. The pier inside the tank, hanger and ceiling support have a great possibility of damage. Stability of the entire structure, however, is not affected by such damage. Although there is also high possibility of the foundation beam and pile foundation being damaged, it is inconceivable that large deformation remains in the damaged elements because they are constrained by the ground. Since vertical bearing capacity seems to be maintained as shown in the experience of past earthquake

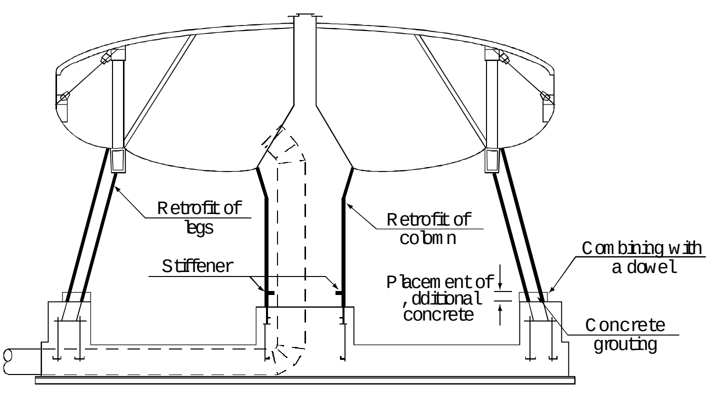


Figure 7: Outline of retrofit

disasters, it was determined that post-earthquake service of the facility and function of the elevated water tank would not be affected. An outline of retrofit drawn is shown in Figure 7.

As a result of an overall analysis of the verification results for individual members, it was found that lining of the center and external supports for retrofit could prevent the structure from undergoing total collapse even during a great earthquake while only partial damage is occurred. The structure thus remains functional after the earthquake assumed in this study.

CONCLUSIONS

This paper reports a case study of seismic diagnosis and retrofit of an existing elevated steel water tank by applying the proposed seismic verification procedure, conducted based on the principles in the revised JWWA Guideline. In seismic diagnoses, level 2 seismic motion, much higher than the conventional seismic motion(level 1 seismic motion), is now considered in addition to level 1 seismic motion. The conventional verification of seismic performance based on allowable stress levels is, therefore, no longer valid for level 2 seismic motion. In this connection, this paper proposed limit states for individual structural elements to meet requirements of seismic performance, and established a practical and rational verification method and criteria accordingly. Verification of seismic performance based on the proposed procedure confirmed that in a case study an elevated steel water tank's seismic behavior, after retrofit by lining of its supports, has no adverse effects on its seismic performance, and that the structure remains fully functional after the earthquake. The authors have carried out similar studies for some existing elevated water tanks which included reinforced concrete structures and steel structures. It was recognized from those study results that the structure-liquid interaction problem could be investigated from a practical point of view because it would affect much the behavior of elevated water tanks.

The ideas and methods presented in this paper are applicable to seismic diagnoses and seismic retrofit of similar structures. The authors would hope this study will help solve similar practical issues in the future.

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