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METHODOLOGIES FOR THE SEISMIC DESIGN OF LIQUID RETAINING STRUCTURES AND THE SEISMIC RETROFIT OF A CASE STUDY WWTP

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Abstract

Water retaining structures form a large portion of the public infrastructure portfolio and provide a variety of functions including irrigation, power generation, water control, flood control and the treatment of water/waste water. Water retaining tanks can consist of steel plates, stainless steel plates or reinforced concrete walls.

Water and Waste water treatment plants (WWWTP) are of particular importance. The dependence of the local public on continuous functioning of WWWTP makes structural damage and their downtime following major earthquakes, a public safety and a health concern. Reinforced concrete tanks are typically designed to remain elastic which would limit cracking and nonlinear deformations, and hence, water leakage. Therefore, in the seismic design of tanks, nonlinear deformations cannot be relied upon to sustain the seismic energy. Consequently, tanks must be designed for the total elastic seismic load without the use of force modification factors. The opportunity for energy dissipation in these structures, through other pathways could be an economical way to design for seismic events, while fulfilling the facilities mandate to remain operational.

Limited guidance is provided to the Canadian engineering community with respect to the seismic design and analysis of WWWTP facilities, especially in the case of liquid retaining structures. These facilities normally utilize concrete tanks for process and storage of liquids. The concrete tanks may be stand-alone structures or incorporated into related building structures. Present Canadian standards do not provide explicit design methods or procedures for non-building structures such as these tanks.

This paper investigates the available Canadian Engineering Standards and compares them to the leading design codes in order to identify an industry accepted standard of analysis. The ACI 350.3-06 analysis methodology is adapted for a Canadian application and is applied in a case study. The case study consists of the seismic retrofitting of large aeration tanks located on the second storey of an existing secondary clarifier designed and constructed in the 1970's. A dynamic analysis is performed considering the vibrational modes of the contained liquid while considering different fixity assumptions at the foundation. Energy dissipation is evaluated at the foundation level considering the effects of soil damping and base slab averaging.

Keywords: Seismic Design; Water Retaining Structures; Energy Dissipation; Dynamic Analysis; Case Study

1. Introduction

The design methods and load requirements prescribed in the 2010 National Building Code of Canada (NBCC 2010) [1] generally relate to building-type structures which are designed for code-specified uses and occupancies. The loads and load combinations have been specifically developed for occupied structures and calibrated statistically to provide a minimum level of performance corresponding to an acceptably low risk of failure that is deemed appropriate for its occupants, regardless of the construction material used or geographic location. The types of loading in non-building structures can significantly differ from building-type structures, as in the present case of liquid retaining structures that are essential to WWWTP facilities. The operational load effects in these structures include dynamic and inertial liquid load effects in combination with a substantial amount of dead mass. This can greatly increase the forces generated during a seismic event when compared to building-type structures.

Design requirements for non-building tank structures are not specifically prescribed in the NBCC 2010 [1] and Canadian engineers are therefore required to look elsewhere for suitable design and analysis methodologies. This especially applies with regard to the seismic behaviour and performance of liquid retaining concrete structures.

The current design practice in the NBCC 2010 [1] for building structures requires designers to resist seismic loading through controlled deformation (rotations and deflections) and intentional energy dissipation using a force-based approach with provision for ductility. Relatively large lateral displacements are permitted under seismic loads; inter-storey displacements of 1.0, 2.0 and 2.5% are acceptable for post-disaster, high importance and normal category buildings, respectively.

The NBCC 2010 Structural Commentary [2] states that building structures that house essential services should remain operational immediately after an earthquake. WWWTPs are normally understood to fall in this class and thereby need to be designed to meet the Post-disaster Importance Category, as a minimum.

The NBCC 2010 [1] requires that Post-disaster structures be designed with a minimum ductility factor of R_d = 2.0 (cl. 4.1.8.10 (2) c.) for the Lateral Force Resisting System (LFRS). For concrete structures, this corresponds to a Moderately Ductile design application under CSA A23.3-04 [3]. For typical walls with a height-to-length ratio greater than 2.0, plastic hinges are expected to form at the base above the foundation. Ductility is achieved by strict reinforcement detailing, particularly in plastic hinge regions, where deformations and controlled yielding of the reinforcing is expected during design ground motions. By contrast, liquid retaining concrete tanks normally have squat-walls (height-to-length ratio less than 2.0) and cannot dissipate significant energy by ductile flexural deformation and rotation. The seismic design of tank walls needs to focus on limiting the deformations and rotations to minimize leakage through cracks, implying near-elastic behaviour. This leads to inconsistency between the design approach of the NBCC 2010 [1], which is force-based and presumes plastic deformations and the requirements of liquid retaining structures to remain operational.

The objective of this paper is to investigate the currently implemented industry-accepted standard of engineering practice for the structural design of WWWTPs in moderate to high seismic zones. Multiple design standards were reviewed and recently constructed WWWTPs in different regions in North America were used as a basis of comparison in determining the accepted structural design methodology. The major design standards are briefly reviewed in combination with available design criteria for some recently constructed structures to identify the methods employed for analysis and design of these representative structures. A case study is presented, wherein the ACI 350.3-06 [4] design requirements are adapted and applied in conjunction with the analysis methodology and guidelines of the NBCC 2010 [1]. The analysis procedure is then applied to a liquid retaining structure located in a relatively low seismic region to determine the expected seismic behaviour. A parametric study is then performed to investigate different fixity conditions and general analysis assumptions for the case study structure.

2. Literature Review

The methodology behind the literature review was focused around currently implemented practices. As the Canadian codes provide little guidance in the design of WWWTPs, the objective was to determine the acceptable design standards for seismic analysis and design of WWWTPs. First, major codes were reviewed to evaluate the industry evaluation of safety and risk related to seismic analysis, response modification factors, importance criteria and allowable drift limits. Then the most relevant code to the case study is used to define seismic loading on the structure.

2.1 Codes and material standards

Major design codes were reviewed to determine the commonly used seismic design variables and the values were compared to the NBCC 2010 [1] procedures.

For the purpose of discussion, only the available standards developed in high seismic areas are presented, as the intent of the review is to determine the latest and most advanced methods of analysis and design. These codes and standards were compared to the NBCC 2010 [1] methods in the attempt to identify shortcomings with respect to the design of the WWWTP. The codes were compared to specifically look at seismic design parameters and analysis methodology and are presented in Table 1.

Table 1 – Code comparison

Design Code	**% of Exceedance**	**Return Period**	**Design Category**	**Importance Factor**	**Response Modification Factor**	**Drift Limit**
NBCC 2010 [1]	2 % in 50 yrs	2500 yrs	Post Disaster	I_E = 1.5	Min R_d = 2	1 %
ASCE 7-10 [5]	2 % in 50 yrs	2500 yrs	Category III	I_E = 1.25	Based on connection (Range between 1.5 to 3.25)	1.5 %
ACI 350.3-06 [4]	2 % in 50 yrs	2500 yrs	Category II	I_E = 1.25	Based on connection (Range between 1.5 and 3.25) Max permissible is 3.25	N/A
CBC 2013 [6]	*	*	Category III	*	Based on Design Class	*
OBC 2010 [7]	*	*	Category III	I_E = 1.25	Based on Design Class	*
IBC 2012 [8]	1 % in 50 yrs	5000 yrs	Category III	I_E = 1.25	*	*
FEMA 750 [9]	2% in 50 yrs	2500 yrs	*	HAZOP	N/A	*
FEMA 369 [10]	2% in 50 yrs	2500 yrs	*	HAZOP	N/A	N/A
NZS 1170.5 [11]	10% in 50 yrs	500 yrs	N.A	N.A	1.0 to 1.5	N.A

**Refers to ASCE 7-10*

2.1.1 Seismic hazard

Design standards for different areas are primarily focused on determining the risk of structural failure for a certain level of ground shaking based on a common design life of a structure. Hazard and risk are often used interchangeably but have very different meanings in structural engineering. A hazard is defined as a type of potential failure (e.g., structural failure, slope failure, liquefaction, etc.), whereas a risk is defined by a certain hazard developing within a specific period of time. The return period is simply based on the percentage of exceedance within the set period of time which is used to describe a certain event. For example, 10% probability of exceedance in a 50 year period yields a return period of 500 year.

In general, the common design life of a building type structure is defined at 50 years for all reviewed codes. In Canada, the spectral response prescribed by code is provided at a level of exceedance of 2% for a 50 year period (2500 year event) and for 4 common periods of vibration (e.g., 0.2s, 0.5s, 1.0s and 2.0s). Values for periods of vibration falling between listed periods can be linearly interpolated. ASCE 7-10 [5] uses an identical return period for design; however, the spectral responses are provided for 0.2s and 1.0s. The New Zealand design standard [11] uses a level of exceedance based on a normal importance factor of 1.0 for a level of exceedance of 10% in 50 year (500 year return period).

2.1.2 Design category and performance objective

The U.S. standards adopt a design category for analysis and design of structures. The category is a function of expected damage at the site location and occupancy requirements. This is related to both the design importance category and the site-specific spectral short period response. In certain designs a geotechnical hazard investigation is required above a design Category C with additional investigation required above a Category D.

The design category can be related to the importance category and performance objectives of the NBCC 2010 [1]. Special requirements are prescribed for higher importance and post-disaster structures. The NBCC 2010 [1] prescribes the use of a LFRS of a minimum of R_d = 2.0 (Moderate Ductility) for post-disaster structures and imposes a 1.0% drift limit.

No minimum ductility requirements are proposed by the ASCE 7-10 [5] and the ACI 350.3-06 [4] for liquid retaining structures; however, response modification factors are suggested to be between 1.5 and 3.25 and a function of the tank anchoring condition. Ground supported tanks are required to be anchored when the short period response value (SDS) is above 0.75 and for importance Category IV tanks (typically associated with high hazard content). The performance of a structure in a specific design category is expected to meet prescribed performance criteria. This revolves around the expected level of damage anticipated and the post event operational impacts of the evaluated structure. The NBCC 2010 [1] Post-disaster category criterion requires that a structure to remain operational after a design seismic event. This is more easily defined for building-type structures, where in order for a building to remain operational, it is required that the occupied area be occupied post-event. However, the criterion does not address or define the operational goals for non-building structures such as liquid retaining type structures.

2.1.3 Importance factors

The recommended importance factor, I_e, varies considerably which reflects the multitude of uses and degrees of importance given to tanks and vessels given by the various authorities and codes. The typical ranges appear to be in the 1.25 to 1.5 value. The Canadian code classifies the design of WWWTPs with a seismic importance factor of 1.5. In the U.S., the usual importance value for WWWTP design is based on a Category III building/structure, which is equivalent to the high importance category of the NBCC 2010 [1] and therefore 1.25. Some municipalities, especially on the west coast, (e.g., Sacramento, CA) use a stricter value of 1.5 with a minimum design Category D for the design of WWWTP facilities. FEMA [9] and [10] suggests determining the importance based on the consequence of failure looking at risk associated to loss of life in combination with economic and social impact (HAZOP). Generally, the selection of an importance factor for tanks implies judgment and higher values can be implemented at the discretion of the designer.

2.1.4 Response modification factors

Response modification factors are incorporated into all of the reviewed standards. The purpose of the factor is to provide a better representation of the actual response induced by the ground shaking. As the structure is subjected to an event, the structure and attached mass generate inertia forces as a result of induced accelerations. Forces generated by the inertial movement are lower than a fully elastic response as a result of energy dissipation. Energy dissipation occurs, even in poorly detailed structures, through material cracking and friction. In design, ductility is used to lower forces on structures by allowing a structure to deform and dissipate energy.

Higher detailing is required to ensure that this deformation can be achieved and is properly controlled. Design standards adopt various modification factors based on the type of LFRS and construction material, and limit their use for different geometric configurations (height) and site conditions. Higher ductility structures result in overall lower applied forces and generally higher lateral displacements. Overstrength factors are based on the expected increase in strength as a result of high strain effects, geometric properties and probable behavior of the material.

2.2 Liquid retaining structures

ACI 350.3-06 [4] is the commonly accepted design standard for reinforced concrete structures, specifically tailored for Water and Waste Water tanks in the United States. This standard was reviewed for concrete tanks for both general design and seismic design requirements.

When liquid contents are subjected to dynamic loading caused by seismic excitation, the behavior is described by two main vibration modes. The determination of these modes and participation to the overall behaviour varies based on parameters such as wall flexibility, fixity at the base and level of excitation. The first vibration mode is impulsive, which is associated with the lower portion of the liquid and essentially acts as additional mass to the system. It is assumed to vibrate at the same period as the structure. The second mode is convective and is associated with the upper portion of the liquid and vibrates with a longer period. This mode is associated with the moving portion of the liquid also known as sloshing. ACI 350.3-06 [4] categorizes tanks in relation to the base fixity and geometric shape. The equations provided by ACI 350.3-06 [4] were used for the calculation of the loads.

3. Risk Based Assessment

A review of the design briefs for Brightwater Regional WWTP [12] and the Lions Gate Secondary WWTP [13] facilities indicate that the design of these facilities was developed based on the Owner's performance objectives for the facilities. Specifically, particular attention was devoted to how the operational or post-disaster performance objectives are defined. Emphasis is placed on the redundancy of key operational elements. Other non-process elements are given less attention as long at their failure does not interfere with the overall performance objectives.

The state of Oregon [7] has developed a type of proactive approach that includes the evaluation of the level of expected damage to their water treatment network in a simulated seismic event. The risk of specific failures were categorized based on higher impact versus lower impact regions by including underlying soils structures, age of infrastructure and proximity to the ocean. From this, a goal setting exercise was performed based on the anticipated damage of the systems. The operational capacity of the network, as well as specific elements, was evaluated and rehabilitation goals and timelines were subsequently defined. In this case, the methodology helped government agencies prepare for and raise awareness on the potential impact of a seismic event on the water treatment systems. Risk based assessment processes could be implemented during the design or evaluation of a WWWTP facility. This would help government agencies have a better understanding of the risks associated with their facilities and develop mitigation strategies to subsequently reduce them. It would also facilitate planning and preparation based on informed decisions. This could include emergency procedures with clear protocols to be implemented post event (mandatory inspections, shutdowns, test procedures, etc.).

4. Case Study

4.1 Background

The case study structure consists of a two-storey secondary clarifier structure with administration constructed in the early 1970. Two large process tanks are located on the second level of the structure. The footprint is generally rectangular with an approximate overall size of 20 m by 75 m and a total above- ground height of the structure as 9 m. The second level contains two baffled tanks, the primary tanks and the aeration tanks both separated into four compartments by baffle walls. The primary tank contains approximately 3.8 m of liquid whereas the aeration tank contains approximately 4.9 m of liquid. Plan view and section views of the tanks are shown in Fig. 1 and Fig. 2. A general deficiency was identified within the structure, this being a section near the middle of the plan bounded by control joints at both its north and south ends and thereby has no shear wall in one direction. In the east-west directions, the LFRS consists of lightly reinforced columns and out-of-plane bending of walls. Preliminary analysis indicated that the columns and out-of-plane stiffness of the walls was inadequate to resist the design seismic forces.

4.2 Seismic hazard

The site class was determined as required by the NBCC 2010 [1]. The geotechnical investigation identified underlying bedrock allowing the application of a Site Class A in the analysis. This site is located in a region of low seismicity. However, it was felt that this type of structure is fairly typical of construction during the 1970's and the seismic response of the structure was also evaluated for a region of moderate seismicity. The spectral response values used for analysis are shown in the following figures. Fig. 3(a) presents the spectral hazard curve for the impulsive response for a region of low and a region of moderate seismicity. Fig. 3(b) presents the spectral hazard curve for the convective response for a region of low and moderate seismicity.

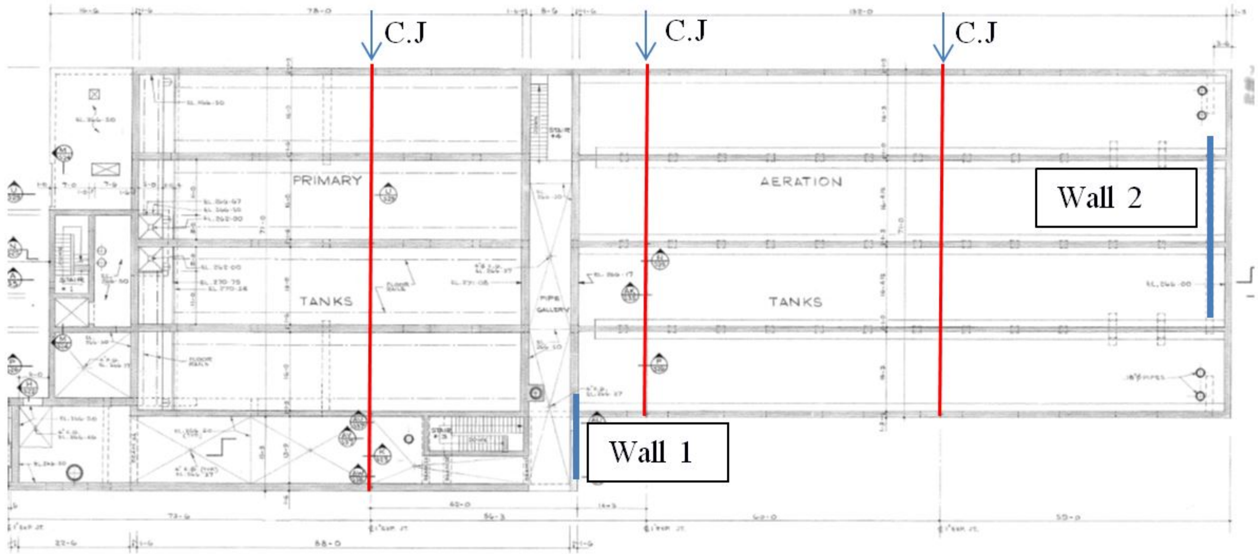


Fig. 1 – Secondary clarifier layout (← N)

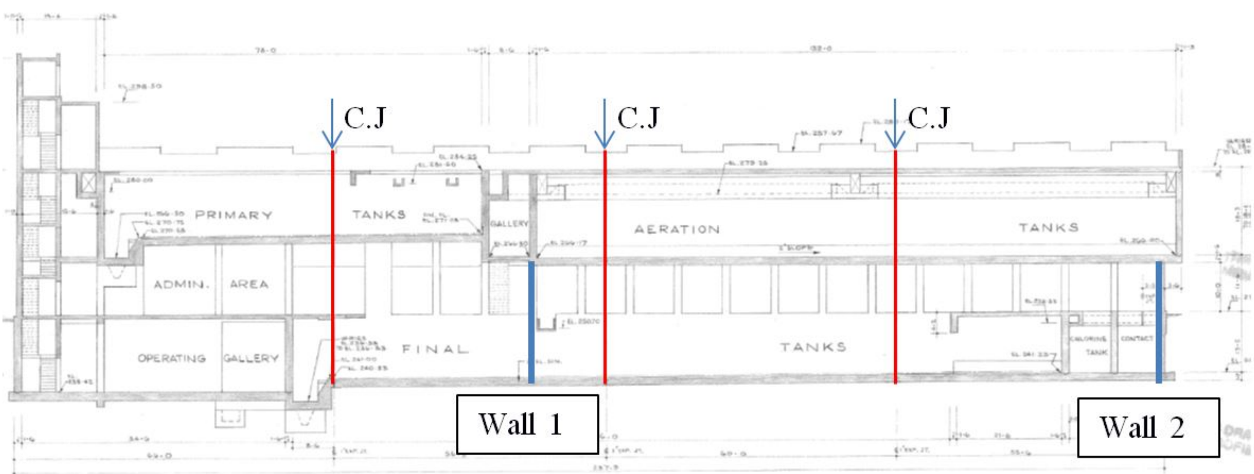
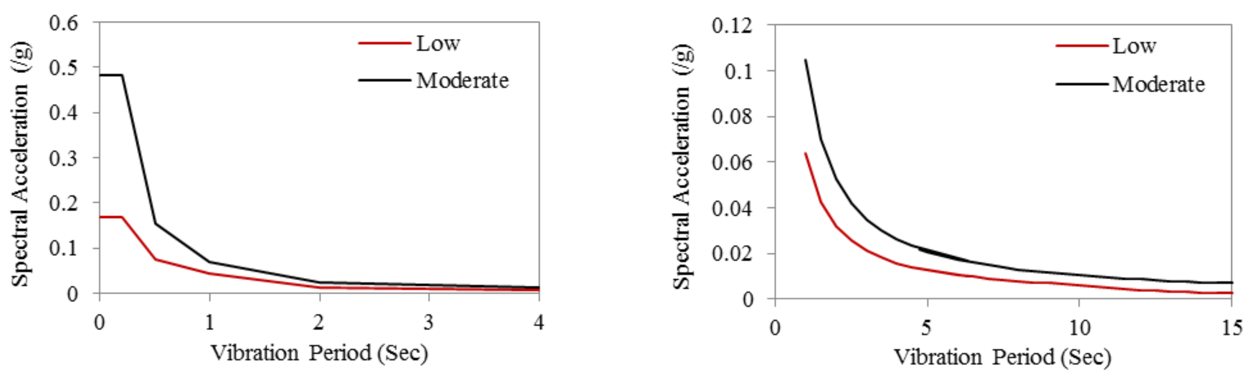


Fig. 2 – Secondary clarifier section



(a) (b)

Fig. 3 – Spectral hazard curves (a) Impulsive mode, (b) Convective mode

4.3 Analysis methodology

The analysis methodology was focused on the seismic retrofit of the tank to reduce the identified deficiency in the structural layout at the expansions joints. General constraints of the site and future expansion limited the retrofit options. At the preliminary stage, the analysis was focused on utilizing the existing structural layout. This was done by stitching the structure together at the expansion joints in an effort to distribute loads to existing walls and determine the overall behaviour and demand. The structure was modeled with shell elements using CSI-SAP 2000.

The evaluation consisted of the following stages: (1) Evaluate the out-of-plane capacity of the existing walls, to ensure that hydro-dynamic and inertial loads could be transferred to the in-plane walls, using the method described in ACI 350.3-06 [4] sections 5.3, (2) Evaluate the behavior of the structure, considering the existing expansion joints, and (3) Evaluate the behavior of the structure with the existing expansion joint removed and the structure positively connected. The development of the dynamic loads for the contained liquid was based on the equations set in ACI 350.3-06 [4]. The procedure was modified and the seismic hazard distribution was adapted based on the NBCC 2010 [1] for the specific site class.

Similar to the NBCC 2010 [1], ACI 350.3-06 [4] does not account for a short period reduction below vibration of 0.2 seconds in the development of its seismic hazard curve. The ACI methodology is based on a maximum considered design earthquake for a specific return period and modified for site specific hazards, seismic detailing and structural importance. For this reason, it was felt that the development of the seismic hazard curve using the NBCC 2010 [1] would sufficiently capture the behavioural characteristics of ACI 350.3-06 [4]. Equations were modified to represent the elastic behaviour of the structure.

The importance factor for post-disaster type structures, as indicated by the NBCC 2010 [1], is 1.5. The maximum response modification factor of the ACI 350.3-06 [4] for pedestal tanks is 2.0. This factor combines ductility and overstrength, and is comparable to the conventional concrete construction factors of the NBCC 2010 [1] of R_d of 1.5 and R_o of 1.3 (R_d R_o = 1.95). In determining the design base shear, the elastic shear is increased by 1.5 (importance factor) and then decreased by 1.95 (ductility and overstrength). The result is a slight decrease from the elastic base shears. However, for the purpose of this analysis, the elastic base shears were used in the evaluation. The loads were calculated using the equivalent static force procedure with additional masses associated with the dynamic modes of the contained liquid. The calculated base shear value was then used for the scaling and calibrating of the dynamic loading.

Once the dynamic loads were calibrated and the loading was determined, the liquid retaining walls were evaluated at the element level for out of plane strength. This step was performed to ensure that the walls could subsequently transfer the loads to the in-plane walls at the opposite ends of the tank. The demand/capacity ratios for the in-plane walls were in the 0.8 range, indicating sufficient capacity to transfer the forces. Once this capacity was verified, the structure was then evaluated as a whole.

In order to include the additional loads from the contained liquid, a fictitious diaphragm level was created and used to represent the impulsive loading on the structure. This additional mass was included in the modal analysis of the structure.

The structure was then evaluated on a per section basis for the individual sections bordered by the existing control joints. In general, the independent structures proved to have sufficient strength to resist seismic loading with the exception of the middle section for east-west loading. Columns and walls within the identified weak section greatly exceeded acceptable levels of stress with demand-capacity ratios in the order of 10. Based on these findings, it was assumed that the control joints would need to be retrofit in order to transfer lateral loads and utilize the in-plane strengths of walls in adjacent sections.

The effects of different expansion joint connection details on the overall behaviour of the structure were investigated. Considering the length of the structure in the north-south direction, it was anticipated that thermal movement could control the design of the new expansion joint connections. It was found that the restraint caused by a fully fixed joint would not allow for the required thermal movement of the structure. It was determined that temperature effects could not be neglected. The detailing of the connection would therefore need to be designed to transfer shear across the joint in the east-west direction and allow for thermal expansion in the opposite north-south direction.

The structure was assumed to be fixed at ground level, when in reality foundation walls and footings extend approximately 4.5 meters below grade. The effects of different fixity assumptions were investigated. The impact of the retrofits presented as demand/capacity ratios for key walls for different fixity assumption at the foundation level. The results of the investigation are presented in the following section.

4.4 Analysis results

The mass associated to the impulsive and convective modes was developed based on the direction of vibration for both tanks and tank dimensions. The dimensions of each tank were based on the baffle walls and independent channels. The associated mass and dynamic properties are defined in Table 2. The estimated mass was calculated based on the ACI 350.3-06 [4] equations. The results are provided in Table 2. It is important to note that ACI does not account for the self-weight of in plane walls.

A Ritz modal analysis was performed on the structure with applied acceleration vectors in both of the major axis with the additional mass incorporated using fictitious diaphragms. The governing vibration modes for each case are presented in Table 2. Only the impulsive mass was included in the modal analysis. The associated seismic coefficient calculated based on the seismic hazards in Section 4.2 and equivalent base shear are also presented in Table 2.

Table 2 – Analysis results calculated based on ACI 350.3-06

Tank and Mode	**Estimated Mass**		**Vibration Period**		**Seismic Coefficient**		**Equivalent Base Shears**	
	E-W (kN)	**N-S (kN)**	**E-W (s)**	**N-S (s)**	**/g**	**/g**	**E-W (kN)**	**N-S (kN)**
Structure only								
Self-Weight (Above Ground)	39,200	26,000	0.12	0.07	0.168	0.168	6,575	4,370
Self-Weight (including Basement)	53,300	46,700	0.16	0.07	0.168	0.168	8,950	7,850
Aeration Tank (weight of liquid)								
Total	38,744	38,744	--	--	--	--	--	--
Impulsive	31,053	5,440	0.18	0.08	0.168	0.168	5,200	930
Convective	10,395	30,820	2.58	11.83	0.025	0.004	260	125
Primary Tank (weight of liquid)								
Total	17,259	38,744	--	--	--	--	--	--
Impulsive	12,157	3,102	0.18	0.08	0.168	0.168	2,050	525
Convective	6,044	13,340	2.54	8.15	0.025	0.009	150	120

4.5 Parametric study

The fixity at the foundation level was investigated considering the effects of the location of fixity (i.e., ground level or bottom of footing level) and the effects of inertial soil structure interaction (soil springs). 5 cases were evaluated including: (1) Structure fixed at ground level (no inertial soil structure interaction effects), (2) Structure fixed at foundation level i.e., bedrock (no inertial soil structure interaction effects), (3a) Structure fixed at the foundation level i.e., bedrock, including the effect of inertial soil structure interaction with spring stiffness

5 MPa/m representative of a soft soil, (3b) Structure fixed at foundation level i.e., bedrock, including the effect of inertial soil structure interaction with spring stiffness 50 MPa/m representative of a stiff soil, and (4) Moderate Seismicity, structure fixed at ground level (no inertial soil structure interaction effects).

The spring reaction was applied at 1/3 of the height of the foundation level. A linear dynamic (response spectrum) analysis was performed for all foundation cases. It was assumed that the springs remained linear and tension was accounted for by reducing the spring stiffness by 50% for both soft and stiff soil conditions. Finally, the effect of increasing the spectral hazard was investigated by using the seismic hazard from a moderate seismic zone. For this case, only the above ground behaviour was investigated. The demand/capacity ratios of all of the walls were evaluated. For discussion purposes, the two walls having the highest demand over capacity ratio are presented below for each load case. The location of the walls is shown in Fig.1. The abbreviations A-F and V refer to Axial Flexure and Shear respectively. The values are presented for the worst case directional loading in Table 3. In essence, in cases 1, 2, 3a, and 3b, the fixity conditions are changed to investigate the fixity condition posing the highest demand on the structure. Changing the fixity conditions does not have a significant influence on the global seismic performance of the structure. This can be attributed to the fact that the structure in consideration is a very stiff structure. In case 4, a moderate seismic hazard is used and the fixity condition producing the highest demand is chosen. Structure will experience the highest demand in this condition. The overall structural behavior was compared in terms of the fundamental structural vibration period and maximum displacements as well. The results are presented in Table 4. The structures deformed shape is shown in Fig. 4.

Table 3 - Demand Capacity Ratio (D/C) of Selected Walls

Wall I.D.	**Case 1**		**Case 2**		**Case 3a**		**Case 3b**		**Case 4**	
	A-F	**V**	**A-F**	**V**	**A-F**	**V**	**A-F**	**V**	**A-F**	**V**
Wall 1	0.550	2.480	0.548	1.712	0.540	1.623	0.540	1.623	1.738	7.232
Wall 2	0.372	2.281	0.334	2.532	0.333	2.529	0.333	2.529	0.954	7.607

Table 4 - Vibration Periods and Maximum Displacements

Cases	**E-W Period (s)**	**N-S Period (s)**	**Max Δ (mm)**
Case 1	0.18	0.08	4.00
Case 2	0.23	0.09	4.80
Case 3a	0.24	0.09	4.90
Case 3b	0.24	0.09	4.90
Case 4	0.18	0.08	7.70

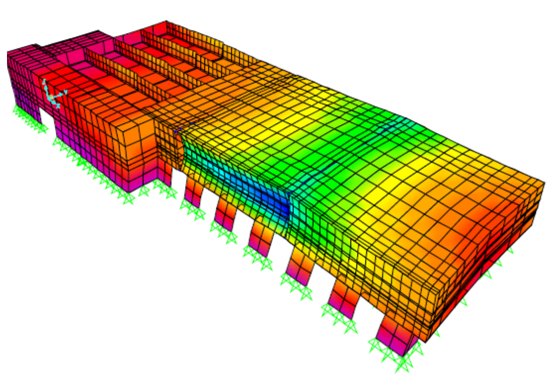


Fig. 4 – Deformed shape (dynamic loading E-W direction)

4.6 Kinematic interaction and foundation damping

The effects of inertial interaction were investigated directly using soil springs indicative of soft and stiff cohesive soil as described above. The effects of kinematic interaction and foundation damping on the seismic response of the soil were investigated using the methods described in ASCE 7-10 [5] and ASCE41-06 [14] for the Seismic Rehabilitation of Existing Buildings. This method accounts for two kinematic effects; base slab averaging and the effect of foundation embedment. The response spectra are multiplied by RRS_{bsa} to account for base slab averaging, and by RRS_e to account for foundation embedment. Considering the dimensions of this treatment facility and the short period value, the RRS_{bsa} factor is approximately equal to 0.84. Therefore, the response spectrum would be decreased by a factor of 0.84 to account for base slab averaging. The embedment of this structure is approximately 14.7 ft. with a period of less than 0.2s. The WWWTP is located in a low seismic zone and founded on bedrock. For these conditions, the RRS_e factor is approximately equal to 0.95. There would be a slight decrease in the RRS_e value if it was situated in a moderate seismic zone but the overall effect would be negligible. Foundation damping based on ASCE41-06 [14] is evaluated. For this structure $\tilde{T}/T$ is approximately 1.3, $\tilde{B}$ is approximately 0.12, compared to 0.05 (or less) normally assumed in structural analysis.

5. Discussion

When the base was lowered to the foundation elevation, the vibration period was found to be 0.23 seconds in the east-west direction and 0.09 seconds in the north-south direction, in comparison to 0.18 and 0.07 seconds respectively when the structure was assumed to be fixed at ground level. This resulted in a period ($\tilde{T}/T$) increase of 27% and 22% respectively. For this structure, the structural vibration period remains in the short period range (just above 0.2 seconds), which only slightly impacts the loading on the structure as a result of the plateau shape of the hazard spectra used in the NBCC 2010 [1] (Fig. 3 (a)). ASCE 7-10 [5] allows for a reduction in the spectral hazard value in the short period range. This reduction is not permitted by the NBCC 2010 [1] and it was not considered in this study.

The results indicate that, for this particular structure, the effects of including soil springs (inertial soil structure interaction) at the foundation level does not significantly impact loading on the walls and vibration period of the structure. This was to be expected as the foundation embedment is low and the structure is relatively stiff. In order for a structure to engage soil springs, a minimum amount of displacement is required. The increased height of the structure used for the inertial soil structure interaction analysis slightly increases the maximum displacement at the top of the structure; this is due to the increase in the structural period. The D/C ratios for flexure and shear are decreased because of the corresponding reduction in base shear. The governing failure mode of the structure is in-plane shear failure of the walls. The structure is not controlled by displacement or flexural yielding.

It can be concluded that the fixed base assumption is conservative. The reductions in the D/C ratios are small for this structure. However, the impact of this assumption would be more significant if the structural period was lengthened past the short period range (0.2s) and moved onto the descending branch of the seismic hazard curve.

The NBCC 2010 [1] suggests the use of a 1.5 importance factor for WWWTPs. The ACI factor ranges from 1.25 to 1.5 based on the hazard of the contained liquid. Water containment is generally considered high importance (Category II). However, the use of importance factors arbitrarily increases the seismic hazard, without giving specific performance requirements. In the NBCC 2010 [1], the only requirement is that the drift be less than 0.01 of the story height.

Simultaneously, the NBCC 2010 [1] allows the use of R_d and R_o factors to reduce the seismic hazard. The use of R_d and R_o factors account for inelastic energy dissipation in the structure. However, the formation of plastic energy dissipation mechanisms suggests permanent deformation and damage, which is inconsistent with the desired performance objectives of liquid containing structures (e.g. to remain liquid tight). The combined use of I_e, R_d and R_o factors creates uncertainty in defining the expected performance of these critical structures. In this study I_e, R_d and R_o were set to 1.0 for simplicity. As the hazard is increased to a moderate seismic zone, it becomes clear that the effects of liquid containment become significant.

The spectral response analysis does not consider the impact of out-of-phase loading of the sloshing portion of the liquid. Spectral analyses are quasi-static and combine the vibration modes of a structure using either the square root sum of squares (SRSS) of complete quadratic combination (CQC) methods. These methods neglect the fact that the convective mode occur out-of-phase to the impulsive mode. If the effects of the convective mode are added to the impulsive mode, the net increase in base shear is negligible. However, if the out-of-phase effects were considered in a time history analysis, they would likely lead to a decrease in seismic loading. Therefore it is considered conservative to ignore that the out-of-phase convective mode affects for the structure.

For WWWTP structures the effects of base slab averaging will normally lead to a decrease in seismic forces. Depending on the type of soil, amount of embedment and seismic hazard zone, the effect of foundation embedment could vary from being almost negligible to a minimum value of 0.453.The effective damping, considering the effects of foundation flexibility, will be increased depending on how the effective period is elongated due to inertial interaction effects.

While the effect of inertial soil structure interaction did not have a significant impact on the structural analysis of the above grade structure, the effect of period elongation due to inertial effects did have an impact on kinematic interaction effects and the effective foundation damping. Due to the size of the structure the reduction factor for base slab averaging is approximately 0.84. The reduction factor for embedment was not as significant, at approximately 0.95. The increase in effective damping considering the effects of period elongation due to inertial soil structure interaction is approximately 40%, effective damping increased from 0.12 compared to 0.05.

The effects of base slab averaging, foundation embedment and increased foundation damping were not accounted for in this analysis. However, if they had been included, the spectral hazard could be decreased by a factor of 0.8 and the effective damping could be increased to 0.12.

6. Conclusions and Recommendations

This paper considers a specific case study; however, certain fundamental components of analysis and methodology were utilized and discussed. The important conclusions with respect to analysis and design of WWWTPs in Canada are as follows:

- Fixity and base elevation assumption will affect the dynamic properties of the structure. For the Canadian Spectral Hazard curve, the assumption of fixity at ground level is conservative as a result of the plateau shape of the curve in the short period range. This should be carefully investigated for spectral hazard models with short period reductions (e.g., ASCE 7-10 [5]).
- The application of I_e, R_d and R_o should be better defined for liquid retaining structures. Ductility requirements of the NBCC 2010 [1] and liquid retaining design methodologies are contradictory in nature. Performance objectives (e.g., drifts and rotation) should be defined for liquid tightness.
- Guidance should be established for the incorporation of the convective mode in dynamic analyses. For this structure, it was determined that the convective was negligible. However, in other cases, the out-of-phase behaviour of the convective mode could result in a damping effect.
- Guidance should be provided to include the effects of soil structure interaction (inertial and kinematic) and increased foundation damping in the analysis and design of WWWTP structures. These factors generally decrease the spectral acceleration values but can lead to increased displacement which could be important for displacement or flexure dominant structures.

The present methodology requires interpretation of non-Canadian standards in a Canadian setting which may lead to unsafe assumptions in compatibility of design and analysis factors. In order to help increase the level of confidence and reduce the risk associated with the design of liquid retaining structures, it is recommended that a Canadian standard for non-building structures be developed and include guidance on liquid containing structures. This document could address the specific risks associated with different types of non-building type structures such as WWWTP facilities, hazardous storage tanks and other non-building type structures with specific vulnerability. The NBCC 2010 [1] focuses on risk in terms of collapse whereas for these non-building type structures, the most significant risk could be leakage, or displacement of sensitive equipment. For these structures satisfying a drift ratio may not adequately address the specific risk. A clear definition of the operational requirements of the NBCC 2010 [1] with respect to WWWTPs could be better established which would provide better guidelines for implementation in the planning, analysis and design of such structures. This would lead to a more uniform design methodology for all WWWTPs and, in turn, a more reliable design. Operational requirements could be defined for both structural elements and process elements. In the case of WWWTP structures, the critical aspect of continued operation may be driven by process related requirements or in meeting regulatory requirements for effluent quality. The document should emphasize proactive designs, proper risk assessment, risk mitigation through strategic planning and layouts. Proactive procedures for post event mitigation should begin at the planning phase of such projects.

The methodology and development of these types of facilities should imply a holistic approach, which would set well established performance objectives between all affected stakeholders. The notion of performance based design has a broader meaning in the context of WWWTP projects. At the structural design level, specific recommendations can be developed for the control of cracking and displacement such that individual wall elements retain liquid after a seismic event. However, addressing the broader notion of performance based design for WWWTPs should include redundancies of process, augmented capacities for emergency storage or other risk mitigating strategies while considering the structural performance after an event. Operational requirements would include regulatory requirements set by environmental agencies (example: less stringent effluent requirements in post event scenarios for a set period of time).

7. References

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