CAPITAL UNIVERSITY OF SCIENCE AND TECHNOLOGY, ISLAMABAD

Numerical Study of Far and Near-Fault Ground Motion Effects on Elevated Water Tank

by

Arif Ahmadzai

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in the

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I want to dedicate this work to my family, who helped me throughout my education. This is likewise a tribute to our best teachers who guided us to go up against the troubles of presence with ingenuity and boldness, and who made us what we are today.

CERTIFICATE OF APPROVAL

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Abstract

Elevated water tanks (EWTs) are essential structures within water network systems in any society for water distribution facilities. In EWTs, significant damage is not acceptable as per Eurocode 8 part 4. In few events of severe earthquakes, such structures were damaged due to unconfinement and water distribution facilities were interrupted which caused difficultly for survivors. Seismic behavior of the elevated water tank can be explored during the strong ground motion records. The confinement can reduce damage significantly. Therefore, this study presents far and near-fault ground motion effects on elevated water tanks by using nonlinear methods for the seismic precise behavior of EWTs.

Elastic and inelastic characteristics of the concrete and reinforcement define separately as a composites material mechanic principle in which coupled in plane/out plane bending and coupled in plane bending-shear nonlinear behavior. EWTs are modelled as both unconfined and confined by considering the stress-strain relationship proposed by Kappos. Seismic deformation capacity of each model is obtained using static pushover analysis. Time history nonlinear analysis is used to predict the seismic deformation demands of the EWTs.

The results indicate that confined EWTs have higher strength and displacement capacities than unconfined EWTs. There is a significant difference between the performance of confined and unconfined EWTs subjected to far and near-fault records. Near-fault earthquake records result higher displacement demands on both confined and unconfined EWTs compare to far-fault earthquake records. The average exceedance ratio of significant damage performance level is higher for unconfined EWTs as compared to that of confined EWTs under far and near-fault records. Unconfined EWTs are not satisfied the significant damage performance level under near-fault records while confined EWTs satisfied it. However, near collapse performance level is satisfied for both confined and unconfined EWTs under far and near-fault records. It is concluded that near-fault earthquake records have significant damage potential on EWTs as compared to far-fault earthquake records, especially on unconfined EWTs.

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Abbreviations

ACI	American Concrete Institute
ANSYS	Analysis System
ATC	Applied Technology Council
CAS	Coupled Acoustic-Structure
CEL	Coupled Eulerian-Lagrangian
ESFP	Equivalent Static Force Procedure
ESM	Equivalent Section Method
EWT	Elevated Water Tank
FEMA	Fedral Emergency Management Agency
IEPA	Improved Effective Peek Acceleration
LD	Limited Damage
NC	Near Collapse
PA	Pushover Analysis
PEER	Pacific Earthquake Research Engineering
PGA	Peak Ground Acceleration
PGV	Peak Ground Velocity
RC	Reinforced Concrete
SAFE	Software Analysis and Forensic Engineering
SAP2000	Structural Analysis Program2000
SD	Significant Damage
SDOF	Single Degree of Freedom
THNLA	Time History Nonlinear Analysis
TSF	Tone per Square Feet
UBC	Uniform Building Code

Symbols

$\mathbf{G}$	Concrete Shear Modulus
$\mathbf{K}$	Concrete Confinement Index
$\mathbf{u}$	Rate of the Descend of the Descending Branch of the Stress-Strain Curve
$\mathbf{R}$	Response Modification Factor
$\mathbf{q}_e$	Allowable Soil Bearing Capacity
$\mathbf{M}_w$	Moment Magnitude
$\mathbf{f}_y$	Steel Rebar Yiled Strength
$\mathbf{f}_c$	Unconfined Concrete Compressive Strength
$\mathbf{f}_{cc}$	Confined Concrete Compressive Strength
$\mathbf{E}_c$	Concrete Modulus of Elasticity
$\mathbf{E}_R$	Rebar Modulus of Elasticity
a,b	Emperical Coeffiecents for Evaluating Confinement Index
$\mathbf{b}_c$	Confined Concrete Core Width
υ	Poisson Ratio
μ	Sliding Coefficient
ε_{cc}	Confined Concrete Strain at Peak Compressive Stress
ε_c	Unfonfined Concrete Strain at Peak Compressive Stress
θ	Chord Rotation
$\mathbf{V}_{max}$	Maximum Base Shear
$\mathbf{s}_w$	Confined Reinforcement Spacing

Chapter 1

Introduction

1.1 Background

Most of elevated water tanks (EWTs) are constructed from reinforced concrete (RC) materials. These structures are constructed for the purpose of water supply from a certain height to provide adequate pressure in water distribution system. Elevated water tanks are extremely important structures for the public services and for the industrial structures. Elevated water tank (EWT) contain huge water mass at the top of elevated tower which is critical for the failure of EWT structure during earthquakes. It tolerates different levels of gravity loads depending on the water level in the tank. Damages of elevated water tanks during earthquakes may endanger water supply for drinking. During past earthquakes in high seismic regions, a large number of these elevated water tanks were severely damaged and collapsed [1]. Far and near-fault ground motions effected elevated water tanks during past earthquakes [2]. Convective and impulsive components of the elevated water tank were investigated with far and near-fault ground motions [3]. It revealed that the response of convective second mode was approximately three times greater under near-fault earthquake records compared to far-fault records.

In this numerical study, the performance of the EWTs is investigated under far and near fault ground motions. First, six elevated water tanks are analyzed on finite element software SAP2000 for the soil profile type SD and seismic zone 4 for seismic source type A, B and C as per Uniform Building Code (UBC-1997) code, and then manually designed as per ACI-318 code. EWTs are analyzed according equivalent static force procedure considering near and far fault factors as per UBC-1997 code. Confined and unconfined concrete properties are considered while conducting pushover and time history nonlinear analysis. Investigated elevated water tanks capacity curves are determined by conducting static nonlinear analysis in the x and y directions. The inelastic dynamic characteristics of the EWTs are considered and seismic displacement demand are calculated under the selected far and near-fault earthquake records. The failure probability for each EWT is comparatively determined and evaluated under the selected far and near-fault ground motions.

Since, elevated water tanks are often used in seismic prone zones, hence seismic performance of the EWTs should be investigated in detail. Some elevated water tanks were heavily damaged or collapsed under far and near-fault ground motions due to insufficient arrangement of transverse reinforcements [5]. Majority of such tanks were unconfined. The confinement had been reported as a good remedial measure for other types of structures. Therefore, there is a need to investigate the comparative analysis of confined and unconfined EWTs under far and near-fault ground motions. According to the best knowledge of author based on the literature review, no study is performed on the comparative analysis of mashroom type elevated water tank under far and near-fault ground motions. In this current work, far and near-fault ground motion effects are investigated to check the seismic behavior of elevated water tank during earthquake motions.

1.2 Research Motivation and Problem Statement

The project aims to investigate the comparison of far and near-fault effects on elevated water tank during strong earthquakes. Far and near-fault ground motions effected on engineering structures especially elevated water tanks during past earthquakes. There is a need to check the behavior of elevated water tank during far and near-fault earthquakes. Elevated water tanks are essential structures and they must remain functional after severe earthquakes, because the failure of the elevated water tank effects public services and fail to control fires after earthquakes. Thus the problem statement is as follow:

Elevated water tanks are essential structures as they provide water for public services, industries, and for the control of fires. It must keep water distribution facilities after severe earthquakes. It is reported that EWTs have been damaged during far and near-fault ground motions, and majority of these have been unconfined. Confinement can save better seismic performance. Therefore, investigation of far and near-fault ground motions effects on elevated water tank having unconfined and confined concrete is important to explore.

1.3 Overall Objective of the Research Program and Specific Aim of this MS Research

The overall objective of the research program is to explore seismic behavior of elevated water tanks during strong ground motions. The specific aim of this MS research work is to investigate far and near-fault effects on elevated water tank during strong ground motions.

1.4 Scope of Work and Study Limitations

Analytical work is done on modelling software of the elevated water tank to investigate far and near-fault ground motion effects. In this research program 30 EWTs are modelled in SAP2000 and 222 analyses are carried out, 6 for static linear, 24 for static nonlinear/pushover and 192 for time history nonlinear. First 6 models are analyzed according equivalent static force procedure as per UBC-1997, four for near fault factors and two for far fault factor and design as per ACI-318. Then these 6 models are divided into 12 models considering confinement of the concrete. The different between confined and unconfined models is in the transvers reinforcement, and longitudinal reinforcements are same. After that, confined and unconfined models are analyzed according to static nonlinear analysis to check the capacities of the EWTs. Time history nonlinear analysis is performed on the confined and unconfined EWTs to determine the seismic displacement demand under consider far and near-fault earthquakes. Exceedance ratios of the EWTs are determined to evaluate the limit states performance levels as per Eurocode 8.

Study limitations include only numerical analysis are carried out in modelling software. The elevated water tanks are analyzed for soil profile type SD and seismic zone 4. Pushover analysis is performed to check the capacities of the EWTs. Time-history nonlinear analysis is performed to check the seismic performance of the elevated water tanks during far and near-fault ground motions. Confined and unconfined EWTs strength and displacement capacities are compared. Near and far-fault ground motions effects are considered for both confined and unconfined EWTs, no experimental work is performed. Substructure is outside of the scope of this work. The study limitation is only superstructure. Therefore, the base of superstructure is fixed.

1.4.1 Rationale Behind Variable Selection

There are many softwares for the analyses of elevated water tanks such as SAP2000, ETABS, ARSAP, etc. Selection of the SAP2000 software, it has the default template of EWT. It can model the EWT in short time. Reason of the selection of pushover and time nonlinear history analyses is to remain functional of EWT during and after severe ground motions, because many elevated water tanks were damaged during severe ground motions.

1.5 Research Novelty, Significance and Practical Implementation

Literature review reveals that the effects of earthquakes on unconfined structural elements are more destructive than confined structural elements during seismic ground motions. The reason for such destruction is insufficient ductility to prevent permanent drift [37]. The significance of this study is to investigate the behavior of confined and unconfined EWTs under far and near-fault ground motions and to ensure the serviceability of EWTs in post severe earthquakes. Furthermore, nonlinear approach has been utilized to establish the capacity curves to validate the seismic stability and performance of EWTs for active seismic zones. Nonlinear approach validates the theoretical approach for possibility of utilization of confined EWTs for the actual infield practice. This may control and reduce the significant damage of EWTs during severe earthquakes. Therefore, the importance of this research is to prevent the significant damage of EWTs during earthquake.

1.6 Brief Methodology

Elevated water tanks having seismic zone 4 and soil profile type SD are modelled in SAP2000 version 21 manually. Different far and near-fault ground motions are taken from Pacific Earthquake Research Engineering (PEER) database and all guidelines of the PEER database are followed for the selection of suitable ground motion records. Total 8 different near and far-fault ground motions are performed on elevated water tanks, 4 for near-fault and 4 for far-fault. First, the elevated water tank is analyzed by using equivalent static analysis procedure as per UBC-1997 and ACI-371R, and then manually design as per ACI-318 and ACI-371R codes. Pushover analysis is carried out to check the seismic behavior of elevated water tanks, different graphs are prepared such as base shear verses displacement. After that, time-history nonlinear analyses are performed on confined and unconfined EWTs under different far and near-fault ground motions to determine the seismic displacement demand. The results are compared in different aspects to evaluate the limited damage, significant damage and near collapse performance levels.

1.7 Thesis Outline

In this thesis there are six chapters which are summarized here below: Chapter 1 illustrates introduction section. It consists of background, research motivation and problem statement, overall objective and specific aim, scope of work, Research Novelty, Research Significance and Practical Implementation, research methodology and thesis outlines.

Chapter 2 briefs literature review comprehensively. It includes background, damages of elevated water tanks during past earthquakes, far and near-fault effects on elevated water tanks, seismicity of Pakistan, performance of elevated water tanks during strong ground motions, numerical approaches for predict elevated water tank behavior, FE methods, softwares for the analysis and design of EWTs and summary.

Chapter 3 explains experimental program. It involves background, description of EWT, material characteristics, modelling, external stability checks, parameters to be analyzed and summary.

Chapter 4 consists analysis and results. It contains background, damage limit states, dynamic characteristics, strength and deformation capacity of elevated water tank, ground motion data and estimation of seismic demand, equivalent single degree of freedom idealization of elevated water tank response, nonlinear dynamic history analysis and performance evaluation and summary.

Chapter 5 compromises dialogue. It consists background, comparison of current research work with previous, guidelines for practical designers and summary.

Chapter 6 covers conclusion and future works.

Bibliography is presented after chapter 6.

Chapter 2

Literature Review

2.1 Background

Elevated water tanks are used to supply water to the consumer at a certain height to provide adequate pressure in water distribution system. These structures provide water to the consumer under hydrostatic pressure which is produced due to its hydraulic head. It is a very important structure of the society compare to other structures. Elevated water tanks are life line structures for the society because people need water to survive. Past earthquakes damaged these structures around the world. If elevated water tank damage during the earthquake the water distribution to the society will be cutoff. Therefore, these structures must remain function after the earthquake. In this chapter brief literature review is performed on the seismicity of Pakistan, damages of EWTs, near and far-fault effects on EWT structures, performance of the EWTs during past earthquakes and finite element methods.

2.2 Damages of Elevated Water Tanks During Past Earthquakes

Recent earthquakes have damaged a number of EWTs in different areas of the world [1]. Occurrence of failures in EWTs due to severe seismic impulsive events in

California in 1994, Northridge earthquake, in Japan in 1995, Kobe earthquake and in Taiwan in 1999, Chi-Chi earthquake. These failures had different reasons such as tank wall buckling failure due to development of excessive compressive stresses in the wall. Piping system failures and anchorage system uplift failures were also observed in those areas. Many elevated water tanks (EWTs) were severely damaged in 2001 in Kutch city of India [2]. Among those damaged water tanks, three elevated water tanks were collapsed, one at near Manfara, second at Chobari and third in Bhachau. Any major damages of elevated water tanks due to natural hazards such as earthquakes would cause serious devastating environmental consequences and economic losses [44]. Many elevated water tanks were damaged strongly during in 1960 Chili earthquake and in 2001 Beijing earthquake [45]. Elevated water tanks torsional failures were observed in past earthquakes such as Keren County in 1952 and Killari in 1993 earthquakes [4]. This problem was established when torsional to lateral natural period ratio is close to 1. Torsional to lateral natural period ratio significantly varied in full tank and empty tank conditions. Fig. 2.1a shows collapsed configuration of the elevated water tank during Bhuj earthquake in India [5]. The capacity of the tank was 265000 liters and during the collapse, the tank was approximately half-full. The collapsed tank was about 20 km away from the epicenter. Thus, Earthquake caused catastrophic failure of the elevated water tank.

Different types of cracks have been observed in the elevated water tank shaft due to earthquakes [5, 6]. Circumferential cracks were the most common type, which was due to flexural action. These cracks were generated in shaft close to foundation where the largest bending moment exist. These cracks (diagonal flexural-shear cracks) appeared in materials with low tensile strength due to shear and bending stresses combination. Diagonal circumferential cracks were the second type of cracks in elevated water tanks which were generated due to bending moment, shear and torsion or the combination of these. Third type of cracks were vertical cracks which were produced due to high compression stress in shaft due to earthquakes. Vertical component of the earthquake produces excess compression stresses in shaft, when these stresses are larger from the strength of shaft, then vertical cracks are produced. These cracks are often visible in the shaft which has poor detailing and insufficient transverse reinforcement. **Fig. 2.1b** and **2.1c** show cracks pattern in shaft of the elevated water tanks. **Fig. 2.1b** shows diagonal circumferential and horizontal cracks in shaft and **Fig. 2.1c** shows vertical cracks in shaft of the elevated water tank. Cracks are dangerous for the elevated water tanks especially at the shaft of tanks as it reduces the capacity of the tank, and may reinforcement corrosion occur.

(a) (b) (c)

Figure 2.1: Damages of Elevated Water Tank, (a) Shaft Supported EWT Collapsed Pattern [5], (b) Diagonal Circumferential and Horizontal Cracks In Shaft [7], (c) Vertical Cracks in Shaft [7].

2.2.1 Near and Far-Fault Effects on Elevated Water Tanks

Numerical study was carried out on hybrid sliding rocking columns and hybrid sliding rocking isolated columns as elevated water tank supporting system to check its effectiveness of different near and far-fault earthquakes [8]. Hybrid rocking columns supported system reduced base shear accordance with far-fault and pulse near-fault ground motions. The combination of isolated hybrid sliding columns and hybrid sliding rocking columns supported system was more effective to reduce structural potential damages. Comparative analysis was carried out on intake water tower tank of different near-fault earthquake records with fling-step pulses, without velocity pulses and forward-directivity [9]. Without scaled near-fault records, intake tower acceleration response with intermediate fundamental time period were strongly associated with Peek Ground Acceleration (PGA) of the ground motions. Larger average PGA like non-pulse records were larger than the average maximum acceleration of intake tower nodes. The tower displacement responses were associated with peek ground velocity (PGV) of near-fault records. Average PGV of the fling-step pulses records were the largest and the tower mean displacement response was also the greatest.

2.2.2 Seismicity of Pakistan

Pakistan and its neighborhood are located in seismically active prone regions. There were numerous earthquakes in the history of Pakistan which were very disastrous such as in 1935 Quetta earthquake had a magnitude (Mw) of 7.4, in 1945 Markan earthquake Mw of 8, in 2005 Muzaffarabad, Kashmir earthquake Mw of 7.6 and etc. [10]. **Table 2.1** shows some major earthquakes occurred in Pakistan between (1935- 2019). Most active tectonic belts for around Pakistan are Himalayan mountain regions [11]. The active thrust belt and fold along the northwestern margins of the Indo-Pakistan plate is divided into two parts, the northwest Himalayan belt and the Suliaman belt. Eurasian and Indian tectonic plates divide Pakistan longitudinally into two parts [12]. Some major disastrous earthquakes in Pakistan were located in the following areas.

Table 2.1: Major Disastrous Earthquakes in Pakistan

Data	**Affected Area**	**M$_w$**	**Depth(km)**
12/25/2015	Gilgit - Baltistan, Khyber Pakhtunkhwa	6.3	212.5
9/24/2013	Awaran District, Balochistan	7.7	14.8
1/18/2011	Dalbandin, Balochistan	7.7	101
10/8/2005	Azad Kashmir, Balakot	7.6	15
2/27/1997	Balochistan	7	10
12/31/1983	Gilgit-Baltistan	7.2	214
11/28/1945	Makran Coast, Baluchistan	8.1	25
5/31/1935	Ali Jaan, Balochistan	7.7	-

- Faults for Karachi and southern Pakistan are, Nagar Parker fault, Ridge Axis fault, Jacobabad fault, Gazabad fault etc. [13].
- Active faults for Balochistan are, Ornach Nal fault for southern regions and Chaman fault (very active fault) for northern regions [14].
- Faults for Peshawar and Khyber Pakhtunkhwa or northern Pakistan are, Punjal fault, Hazara Kashmir syntax fault, Jhelum fault and etc. [10].

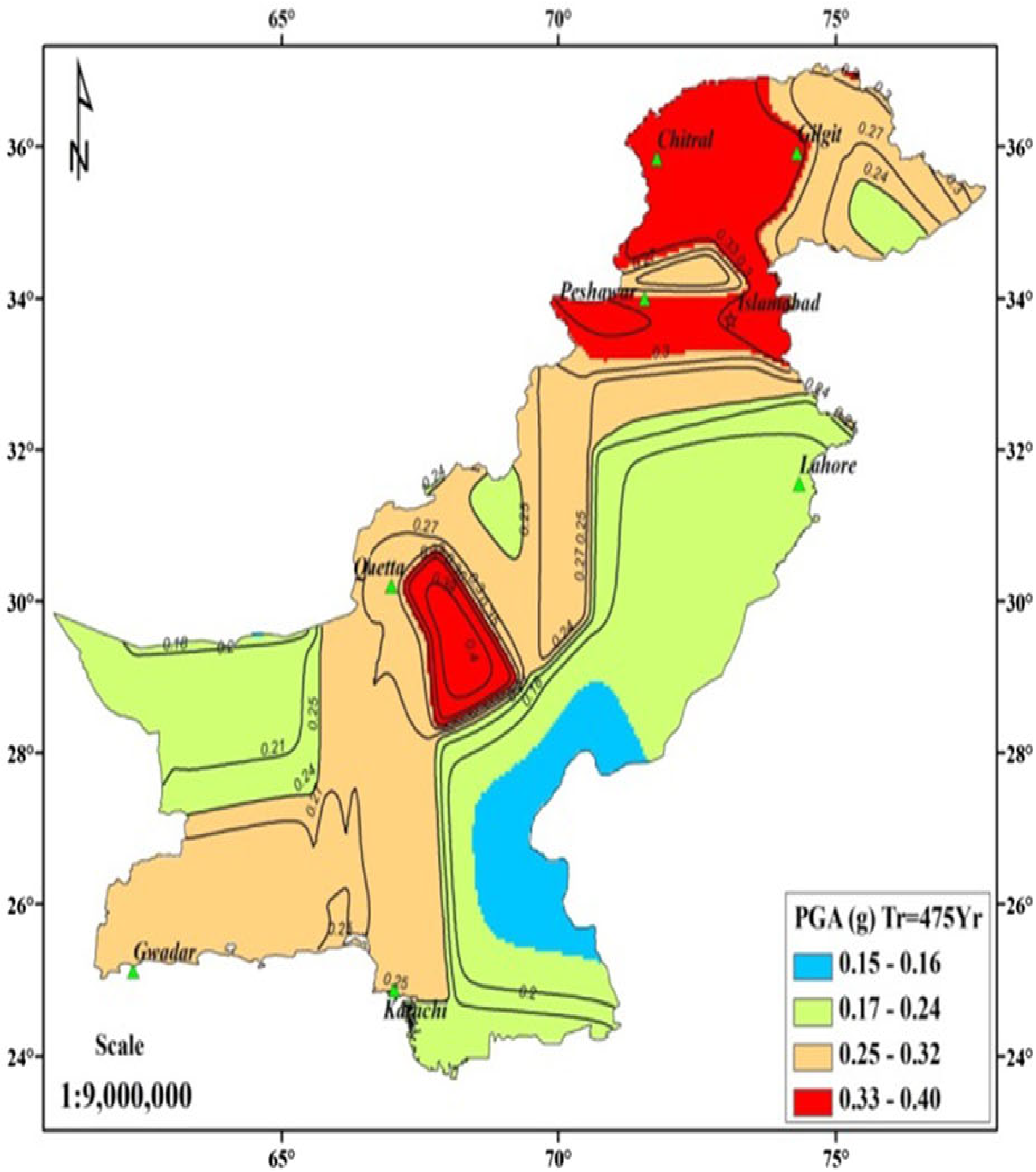

FIGURE 2.2: Seismic Probabilistic Hazard Map for Pakistan of PGA in "g" with Return Period of 475-Year [15].

Pakistan is geographically located in the most seismic prone regions in the world. An investigation was done on probabilistic seismic hazards assessment of Pakistan [15]. It showed that the higher seismic was present in those regions where active tectonic plates were located such as the Hindu Kush, Makran zone and the active fold that enters form southeast direction of Pakistan (includes; Sulaiman, Kirthar mountains and Himalaya ranges). Fig. 2.2 shows seismic hazard map of Pakistan based on PGA values corresponding to 475 years return period for different part of the country. PGA corresponding to 475 years has been the standard parameter used by seismic hazard studies in Pakistan [15]. The highest seismic hazard of PGA value is 0.4 g observed in Pakistan for the Sulaiman regions corresponding to 475 years return period. For the Islamabad and its surrounding areas, the seismic hazard of PGA value is observed between (0.33 - 0.4 g) corresponding to 475 years return period. Acceleration at the ground surface in the Rawalpindi-Islamabad area is in the range of 0.92-0.94 g, 0.71-0.77 g, 0.59-0.65 g, and 0.4-0.48 g for the return period of 2475 years, 975 years, 475 years and 150 years, respectively, where g is the acceleration due to gravity [16]. Most of the Rawalpindi-Islamabad area has moderate to high acceleration values based on the shear wave velocity and site response analysis of the site.

According to available data, Pakistan has different level of earthquakes form moderate to high level. It has great number of light earthquakes of M_w <5.5, frequently moderate earthquakes of Mw are between (5.5 -6.5), and a small number of strong earthquakes of Mw>6.5. **Fig. 2.3** shows the epicentor distributions of strong earthquakes of Mw>6.5 in different areas of Pakistan. It clearly showed that, in the history of Pakistan there were located a number of large earthquakes with magnitude grater than 6.5.

Diasotrous earthquakes were also located in the hisoty of Pakistan with magnitude greater than 8. Near to Isalamand-Rawalphindi area, a number of large earthquakes were located with magnitude greater than 7.5. Some of them occurred recently such as Azad Kushmir earthquake in 2005. Most seismic prone areas of the country are some part of the Bulochistan state, Azad Kashmir area and some northern part.

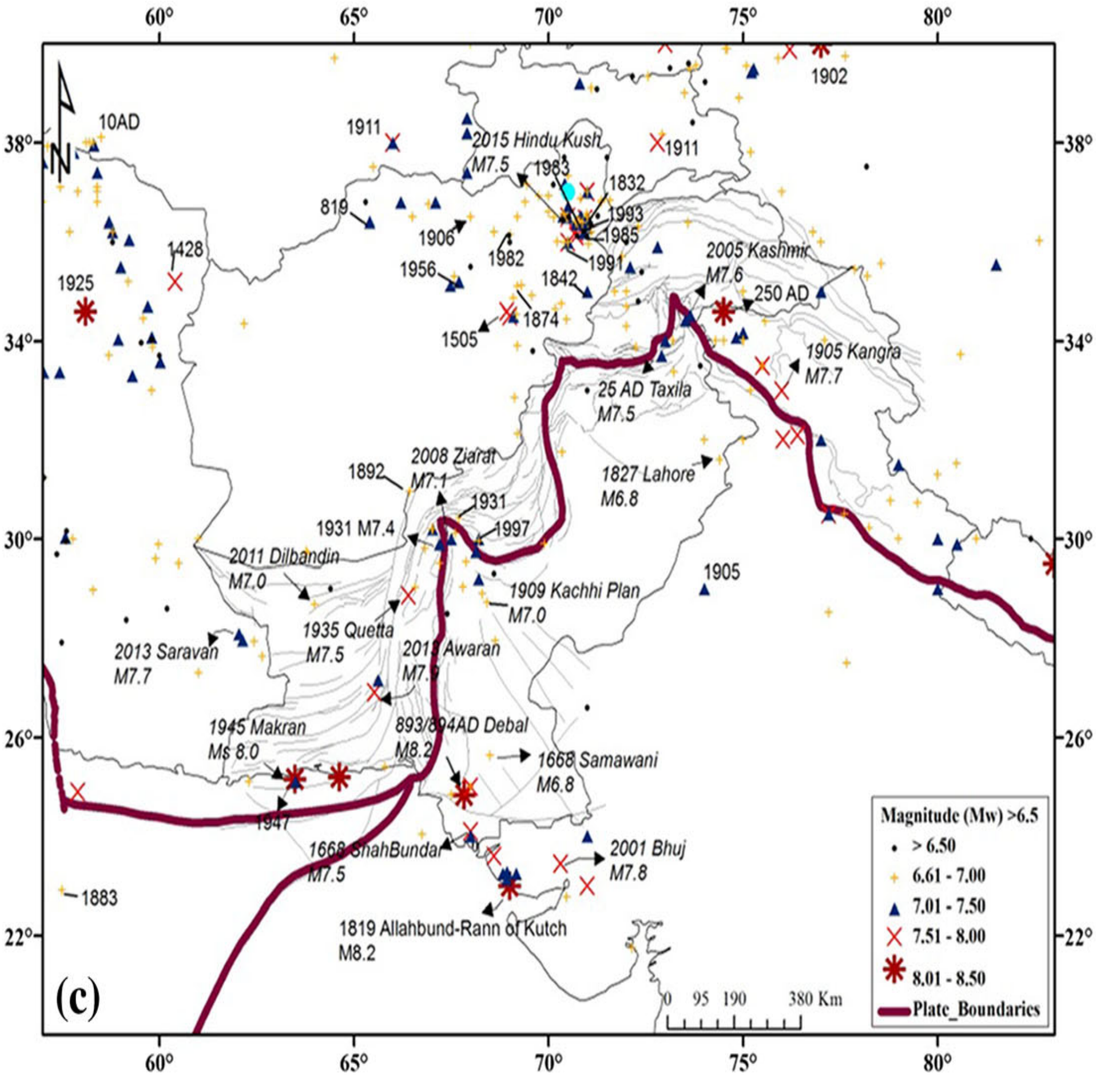

FIGURE 2.3: Earthquakes Magnitude Greater Than > 6.5 Epicenter Distribution in the Region [15].

2.3 Performance of Elevated Water Tanks During Strong Ground Motions

There are limited research studies in literature which investigated the behavior of elevated water tank under far and near fault ground motions. The performance of the elevated water tank is related with material characteristics and supporting system. The supporting system of the EWTs has significant rule on the seismic performance of the overall tank. A numerical study was carried out on different sizes of pedestal supported elevated water tank systems with different severe ground motions [17]. It revealed that light and medium size elevated water tanks had better seismic performance compared to heavy elevated water tanks. The overall performance of light and medium size pedestal supported elevated water tank system were good and it had 40% less lateral deformation of its ultimate deformation capacity. Elevated water tank size was a critical parameter and it effected the performance of the tank [18]. Result showed that same tank size, shorter tank demonstrated higher maximum base shear (V_{max}) compared to taller tank. Same EWTs pedestal height but different tank sizes had difference performance in same seismicity zones. Heavy EWTs had more lateral deformation and experience more damages compared to medium or light EWTs with same pedestal height.

Numerical study was performed on vertical isolated system to improve the seismic behavior of EWTs [19]. Result showed that the proposed isolated system decreased overturning moment significantly. However, the proposed system was more effective for slender tanks compared to broad tanks and it increased the effective damping of the EWTs. Influence of mechanical inerter system was studied of the EWTs to check the seismic response of base isolated system [20]. It revealed that the mechanical inerter systems significantly affected the seismic response of EWTs and that impacts were varied with changes of aspect ratio of the tanks, external excitation frequency and systems parameters.

Numerical study was carried out on EWT to check various water level effects on the seismic response of EWT and sloshing water [49]. It revealed that the tank walls significantly affected by water depth, mass ratio, oscillation frequency and height to diameter ratio. An investigation was done on elevated water tank to check the sloshing water effects during earthquake excitations [50]. It showed that considerable oscillation pressures at walls of the tank near to free surface compared to deeper surfaces in the tank. The maximum free surface displacement was the most raised when the tank was least filled. The performance of EWTs are different in different soil conditions. On the flexible and rigid bases of EWTs, experimental investigation was carried out on the prototypes of EWT using shake table [21]. Actual seismic records of the Wellington city of New Zealand were used. The result demonstrated that the prototype directly placed on the shake table (rigid base) increased the compressive axial stresses in shells. In 95% cases in flexible soil (provided sand between shake table and prototype) top displacements were higher. In 77% cases in flexible soil, elevated water tank walls acceleration was higher.

2.3.1 Effects of Confinement in RC Walls

Concrete confinement by transverse reinforcement significantly improved the ductility and strength of the RC elements [22,23]. A numerical investigation was done on concrete shear walls confined by transverse reinforcement [24]. Results showed that concrete confined by transverse reinforcement in a shear wall compressive zone significantly increased ductility and strength of the wall. Experimental research was conducted on reinforced concrete shear walls to study the effect of confinement on the performance of wall to artificial earthquakes [25]. Results revealed that the efficiency of confinement highly depended on the arrangement of transverse reinforcement. Simply increased the transverse reinforcement in specimens did not produce a large degree of confinement. An investigation was done on existing reinforce concrete shear wall building to check the performance of the wall [47, 48]. It showed that confinement of the boundary element of shear wall significantly improve ductility and strength of the wall.

2.4 Numerical Approaches for Predict Elevated Water Tank Behavior

There are limited research studies which investigated the time history nonlinear seismic response on elevated water tanks. Numerical study was performed on elevated water tanks to investigate the seismic excitation using finite element procedure [26]. In the investigation, two approaches were used, coupled Eulerian-Lagrangian (CEL) and coupled acoustic-structure (CAS) of the finite element method (FEM) by using Abaqus. These approaches were used to study the fluid structure interaction behavior of the water tanks. Ratios of sloshing peak height

to the height of water in the tank were obtained in rectangular tank 0.046 and 0.06 for CEL and CAS approaches, respectively. Whereas, for cylindrical tank these ratios were 0.039 and 0.0425 for CEL and CAS approaches, respectively. Water hydrostatic pressure was analyzed using FEM for composite tanks; i.e. concrete and steel [27]. A FEM for composite tanks accounts both material and geometric nonlinearities. By including nonlinear models, the material nonlinearity was considered for both concrete and steel. Composite conical tank was analyzed by using Equivalent Section Method (ESM) and FEM for composite method. It revealed that both methods ESM and FEM for composite, predicted forces and stresses separately at the steel shell and concrete wall. Displacement obtained by FEM for composite were significantly larger than ESM method because FEM for composite accounts concrete cracking. Both ESM and FEM for composite methods predicted the failure of steel shell. Forces, stresses and maximum displacement were obtained along the vessel height in both methods and it were identical.
Seismic response of the elevated water tank using slits in RC shaft was studied by FEM approaches with respect to soil type and compared seismic behavior to the solid EWT [28]. Result revealed that the slits width in RC shaft tower affected significantly on the stiffness and failure mode of EWT. Stress concentration at the base of RC shaft was reduced and stresses distributed uniformly along the height of shaft by using slits in the RC shaft. In softer soil when increased slits width decreased the ability of EWT to withstand earthquakes. Because increased the thickness of slits increased the ductility and reduced stiffness in RC shaft. For softer soil stiffer RC shaft was more appropriate to withstand earthquakes. Slits in RC shafts for EWT were appropriated for soil profile type A, B and C based on Eurocode 8. However, for soil profile type D slits in RC shafts of the EWT were not appropriate. Soil profile type A (rock) was the most favorable soil for the construction of EWT and soil profile type C was the least favorable soil.

2.4.1 Finite Element Methods

The dynamic characteristics of rigid square and cylindrical elevated water tanks with manholes (concentric openings) were studied using approximate discrete model and finite element method [29]. The study was comparison and performed on complete soil-structure interaction effects and fixed base conditions. It showed that the geometry of water tank had a strong effect on the accuracy of the sloshing modes. The effects of sloshing modes were different in circular and square water tanks. Seismic performance one of the oldest EWT in Florence with RC staging structure was analyzed by using finite element simulation [30]. There were irregular braces on the three upper staging level and there was no bracing at the first level of staging. Time history analysis was carried out in detail. At the first staging level, EWT columns showed unsafe response conditions. Elevated water tanks uplift mechanism under strong lateral loading was examined by using three dimensional finite element models [46]. Pushover analysis was conducted to define the moment rotation relationship of uplifting tanks resting on rigid foundation. Advance retrofitting and seismic assessment on two heritage RC elevated water tanks were studied by using finite element method [31]. These two EWTs (RC frame staging supported and shaft staging supported system) were constructed in Florence between 1920 and 1930. These water tanks performed their functional up to now. Fluid dynamic behavior of these EWTs were studied by using finite element three dimensional discrete schematization. It revealed that frame staging EWT highlighted numerical collapse on the frame structure and excessive tensile stresses developed in the shaft supported structure.

2.4.2 Softwares

There are many softwares in literature that analysis the behavior of elevated water tanks against lateral load such as fluid structure interaction (impulsive and convective part of water), soil-structure interaction etc. OpenSees was one of the software which investigated the supporting systems of EWTs such as hybrid sliding rocking columns and hybrid sliding rocking isolated columns [14]. EWTs seismic response of the base isolated structures were analyzed by using 3D-BASIS-ME-MB program to check water tanks behavior under near-fault ground motions [32]. Elevated water tanks shaft staging supported system was modelled using SeismiStruct software: version 6 to check the seismic demand of EWT load-resisting members [33]. Base isolation technology of the EWT was analyzed using finite element software SAP2000 to check the seismic behavior of base-isolated elevated water tanks [34]. Storage tanks buckling above from ground was studied using LS-DYNA finite element software to check fragility assessments of the storm surge [35]. There are a number of softwares in literature which are analyzed the fluid-structure interaction of the elevated water tanks. ANSYS is one of the software which is analyzed fluid-structure interaction effects of EWTs. Water-structure interaction effects were analyzed using ANSYS software to check sloshing of water free surface and flexibility of tank walls [36].

2.5 Contribution of this Study

The novelty of current research work assists in its contribution towards the significant damage performance level of EWTs under far and near-fault ground motions. The identified research gap is to resolve seismic functionality aspect of EWTs after severe earthquakes in active seismic zones which is a serious concern. Furthermore, nonlinear approach also validates the theoretical approach for feasibility of utilization of confined EWTs for the actual infield practice. This may control and reduce the drastic failure of EWTs during severe seismic activities and save the previous human lives.

2.6 Summary

Brief literature review showed that disastrous earthquakes were located in the world. It showed that these earthquakes severely damaged elevated water tanks. Various types of cracks were observed in EWTs such as circumferential and vertical cracks near to the base of pedestal. The performance of elevated water tanks is related with material characteristics and supporting systems. Supporting system of the elevated water tank significantly affected the performance of elevated water tank. Soil condition has different effects on the performance of elevated water tank. Performance of the EWT at flexible base has differ from the medium or rock base because the difference between the rigidity of EWT and flexibility of the base. Near and far-fault earthquakes have different characteristics and their effects on elevated water tanks are different.

Chapter 3

Experimental Program

3.1 Background

Elevated water tanks are essential structures of the society and it must be functional after the earthquake. In these structures water flows to the consumer under the gravity and no power is required for the supply of water. Different softwares are used for the analysis of elevated water tanks i.e. SAP2000, ANSYS, LS-DYNA and many others. SAP2000 is one of the softwares which is used for the superstructure analysis of elevated water tanks. Usually, designers take governing moments from the software and perform manually design by considering the minimum and maximum requirements of the code especially for shell elements. In this study, the superstructure analysis is carried out on SAP2000 software according to equivalent static force procedure as per UBC-1997 and ACI-371R. The design is carried out manually by considering the ACI-318 and ACI-371R codes maximum and minimum requirements. The substructure of the elevated water tank should be analyzed on another software such as finite element SAFE software. Substructure of the elevated water tank is not part of this research program.

3.2 Description of Elevated Water Tank

The mashroom type of EWT in **Fig. 3.1** is considered in this research study.

The capacity of the tank is 50000 gallons. The minimum hydraulic height of the EWT is 60 ft and the total height of the elevated water tank from raft top level to the mumty top roof is 88.5 ft. Center to center diameter of the cylindrical shaft, bowel outer wall and cylindrical mumty are 11 ft, 37 ft and 7 ft, respectively. The thickness of the cylindrical shaft is 18 in and thickness of the bowel elements i. e. base slab, inclined slab, outer wall, inner wall, top slab and mumty slab are 36 in, 18 in, 12 in, 12 in, 9 in and 6 in, respectively.

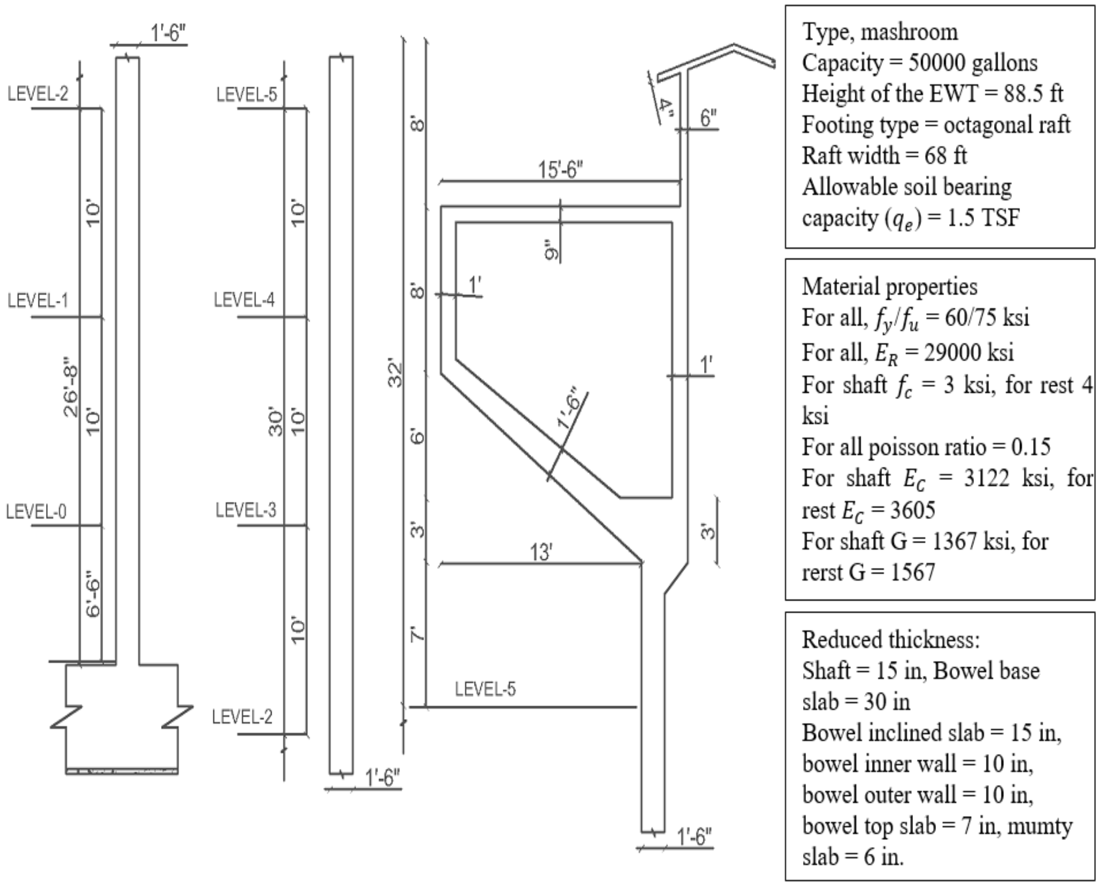

FIGURE 3.1: Layout of Elevated Water Tank and Related Information

Reduced thicknesses of the reduced EWTs are shown in the box of **Fig. 3.1**. Strength of concrete (f_c) for the shaft and bowel elements are 3 and 4 ksi, respectively. The yield strength (f_y) and ultimate strength (f_u) of the steel reinforcement in tension are 60 and 75 ksi, respectively. Poisson ratio (υ) for concrete and steel reinforcement is 0.15. Modulus of elasticity (E_C) for the 3 and 4 ksi concrete are 3122 and 3605 ksi, respectively, and for the steel reinforcement (E_R) is 29000 ksi. Shear modulus (G) for the 3 and 4 ksi concrete are 1357 and 1567 ksi, respectively.

Allowable bearing capacity of the soil is 1.5 TSF and octagonal raft width is 68 ft considered. Shaft of the EWT is divided into three parts on the basis of transverse and longitudinal reinforcement configuration. Part-1 is from raft top to level-1, part-2 is from level-1 to level-4 and part-3 is from level-4 to the bottom of bowel.

Table 3.1 shows material properties of concrete and reinforcement. For shaft and bowel elements 3 and 4 ksi concretes are used, respectively. The modulus of elasticities (E) of 3 and 4 ksi concrete are 3122 and 3605 ksi, respectively. Modulus of rigidities (G) of 3 and 4 ksi concrete are 1357 and 1567 ksi, respectively. The Poisson ratio is 0.15 for both 3 and 4 ksi concrete. Grade 60 steel rebars are used for both longitudinal and transverse reinforcements. The yield and ultimate strength of grade 60 rebars are 60 and 75 ksi, respectively. Grade 60 rebar, modulus of elasticity and modulus of rigidity are 29000 and 12600 ksi, respectively and Poisson ratio is 0.15.

Table 3.1: Concrete and Reinforcement Properties

Concrete/ Steel Type	**Compressive/ Yield Stress (ksi)**	**Ultimate Stress (ksi)**	**Modulus of Elasticity (ksi)**	**Modulus of Rigidity (ksi)**	**Poisson Ratio (-)**
Shaft Concrete	3	2.55	3122	1357	0.15
Bowel Concrete	4	3.4	3605	1567	0.15
Grade 60 steel	60	75	29000	12600	0.15

3.2.1 Material Characteristics

Cylindrical concrete shaft is the member in mashroom type elevated water tank for resisting lateral and vertical loads. It should be carefully designed to provide sufficient strength and adequate ductility to avoid brittle failure under strong lateral loads, especially during an earthquake. For the enhancing ductility of concrete walls, concrete of the cylindrical wall in compression zone should not fail before flexural reinforcement reach to yielding [24]. Concrete confinement by transverse reinforcement prevent longitudinal reinforcement from bucking failure and keep the core concrete from spalling.

Therefore, confinement of the concrete at the wall base by utilization of transverse reinforcement significantly increases the compressive strain of the concrete at which concrete losses the strength and delays the concrete compression failure. It helps the wall to fail in ductile manner. Concrete is very weak in tension because it is a brittle material. A number of tests by different researches showed that concrete confinement by transverse reinforcement increased both ductility and strength significantly [38]. Based on the test results, Kappos developed the relationship of stress-strain for the confined concrete as shown in **Fig. 3.2** [22]. Confined concrete is the concrete in which confinement index K>1, and unconfined concrete is the concrete in which confinement index K=1 [22,43]. According to the Kappos result, confinement of the concrete by transverse reinforcement increased the compressive strength of the concrete by a factor of K and the corresponding strain at the peak compressive stress by a factor of K2 as per the following equations:

$$f_{cc} = K f_c \tag{3.1}$$

$$\varepsilon_{cc} = K^2 \varepsilon_c \tag{3.2}$$

Figure 3.2 shows stress strain relations which proposed by Kappos. In graph, y-direction represents stress and x-direction represents strain. In which (K) is a confinement index, (f_{cc}) and (f_c) are the confined and unconfined compressive strength, (ε_{cc}) and (ε_c) are the strain values of confined and unconfined concrete at peak compressive stress, respectively. The value of K dependent on the confined reinforcement yield strength (f_y), unconfined concrete compressive strength (f_c) and confined reinforcement volumetric ratio ρ_w). For the value of (K) Kappos proposed equation-3.3. In which a and b are the empirical coefficients and functions of the confined reinforcement layout. The valuses of a and b are highly depending on the arrangement of transverse reinforcement. For the simple arrangement of transverse reinforcement these values for a and b are considered 0.55 and 0.75, respectively which are used in this study.

$$K = 1 + a(\rho_w \frac{f_y}{|f_c|})^b \tag{3.3}$$

$$u = \frac{0.5fcc}{0.75\rho_w\sqrt{\frac{b_c}{s_w}} + \frac{3+0.29|f_c|/k}{\frac{145|f_c|}{K}-1000} - \varepsilon_{cc}} \tag{3.4}$$

$$\varepsilon_{sp} = 0.012 - 7 \times 10^{-7} f_c \tag{3.5}$$

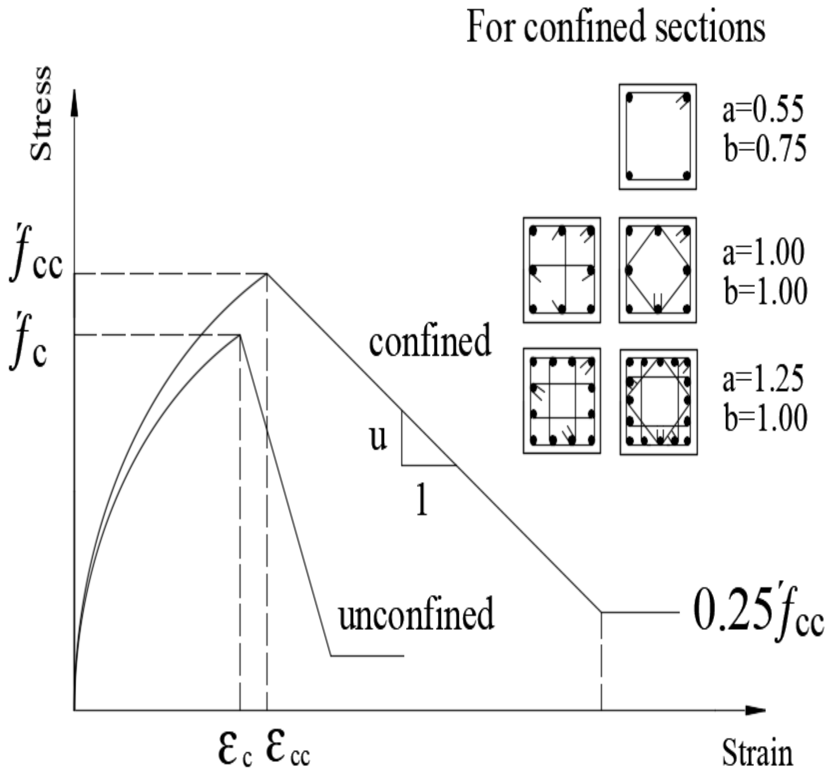

FIGURE 3.2: Confined Concrete Stress-Strain Relation Proposed by Kappos [22].

For the stress-strain curve ascending branch Kappos used a parabolic curve and a straight line curve for the descending branch at a rate of (u) per unit strain, as (u) is given by equation-3.4. (s_w) is the confined reinforcement spacing and (b_c) is the confined concrete core width. Equation-3.4 is applicable for both confined and unconfined concrete when value of K=1. Ultimate stress is equal to 0.85 f_c. The failure strain for unconfined concrete is obtained by using equation-3.5 proposed by Collins and Michells 1994 and failure stress is zero for unconfined concrete [43]. Confined concrete stress strain is caluculated as per the method which is proposed by Kappos. For confined concrete, peak compressive stress is calcuated by multiplying confinement index into concrete stress. The confinement index value is obtained k=1.05 by using **equation 3.3**.

The peak compressive strain is calculated by multiplying K^2 into strain at the peak compressive stress. **Table 3.1** shows confined and unconfined concrete stress-strain relationship. The failure strain for 3 ksi confined and unconfined concrete are 0.015 and 0.009, respectively. For 3 ksi confined concrete, the failure strain is 67% greater than unconfined concrete. This is a big difference between confined and unconfined concrete failure strain. The failure stress of 3 ksi confined concrete is 0.75 ksi. However, the failure strains of the 3 ksi confined and unconfined concrete are 0.015 and 0.009, respectively. For 4 ksi unconfined concrete, the peak and ultimate stresses are 4 ksi and 3.4 ksi, respectively, and ultimate and failure strains are 0.011 and 0.009, respectively. Confined concrete is only considered for shaft of the EWT and rest elements are considered unconfined concrete.

Table 3.2: Confined and Unconfined Concrete Stress-Strain Relationship

Concrete Type	**Peak Stress (ksi)**	**Peak Strain (-)**	**Ultimate Stress (ksi)**	**Ultimate Strain (-)**	**Failure Stress (ksi)**	**Failure Strain (-)**
Unconfined concrete	3	0.0018	2.55	0.007	0	0.009
Confined concrete	3.16	0.0021	2.55	0.008	0.75	0.015
Unconfined concrete	4	0.0019	3.4	0.009	0	0.011

Figure. 3.3 shows confined and unconfined concrete stress strain graphs. The y-direction represents stress in ksi and x-direction represents strain which is unitless. The difference between 3 ksi confined and unconfined concrete is clearly shown in graphs. The 3 ksi confined and unconfined concrete peak compressive stresses

are 3.16 and 3 ksi, respectively. The peak and ultimate strains of 3 ksi confined concrete are 0.0021 and 0.008, respectively. The failure strains of 3 ksi confined and unconfined concrete are 0.015 and 0.009, respectively. The 4 ksi unconfined concrete peak compressive stress and strain are 4 ksi and 0.009, respectively. The failure strain of 4 ksi concrete is 0.0011. The difference between failure strain of confined and unconfined concrete is clearly shown in graphs.

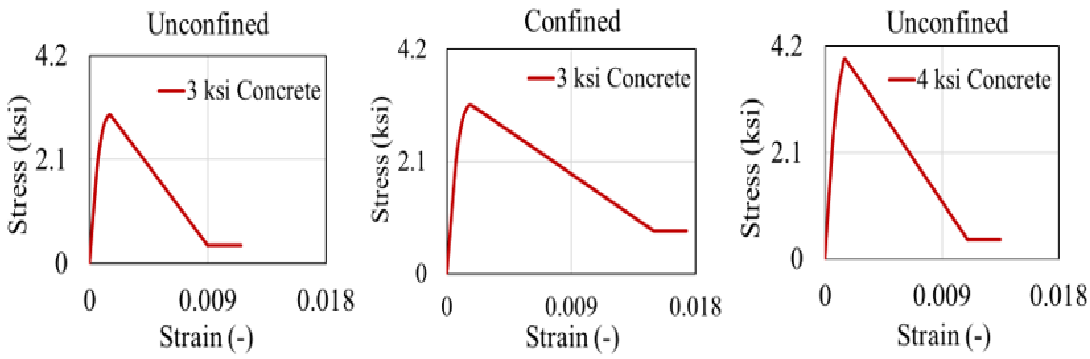

FIGURE 3.3: Confined and Unconfined Concrete Stress Strain Graphs

3.3 Modelling

Modeling is done by using finite element software SAP2000. Total 30 models are modelled and 222 analyses are carried out in this research, 6 for static linear, 24 for static nonlinear/pushover and 192 for time history nonlinear. First 6 models are analyzed according equivalent static force procedure. Then 24 models are analyzed according pushover analysis to check the capacity of EWT. And, remaining 192 models are analyzed according time history nonlinear procedure under far near-fault records as shown in **Table 3.3**. First, A_S, B_S, and C_S models are analyzed according equivalent static force produce for soil profile type SD and seismic zone 4 as per UBC-1997. A_S and B_S models are analyzed for the near seismic source factors and C_S model is analyzed for the far seismic source factor as per UBC-1997 code. A_S, B_S and C_S models are analyzed for seismic source type-A, B and C, respectively. Seismic source types A, B and C are the seismic sources whit seismic magnitude greater than 7, between (6.5-7) and less than 6.5 as per UBC-1997, respectively.

Then, analyzed models are manually designed as per ACI-318 code and it is ensured that all the designed sections fulfill all the ACI-318 maximum and minimum requirements. Basically, these three designed models A_S, B_S and C_S are divided into six models by considering confined and unconfined concrete. A_S is divided into A_C and A_{UC}, B_S is divided into B_C and B_{UC} and C_S is divided into C_C and C_{UC}. After that, Pushover analysis is carried out on the designed sections to check the capacities of the confined and unconfined EWTs. A_C, B_C, and C_C are analyzed for the confined concrete and A_{UC}, B_{UC}, and C_{UC} are analyzed for the unconfined concrete. Then, A_{NLC}, B_{NLC}, and C_{NLC} are analyzed according to time history nonlinear analysis on confined sections, and A_{NLUC}, B_{NLUC}, and C_{NLUL} are analyzed according to time history nonlinear analysis for unconfined sections to check the performance of elevated water tanks. In time history nonlinear analysis different far and near-fault ground motion records are considered.

3.4 External Stability Checks

External stability checks of the elevated water tanks are applied as per UBC-1997 code. Three types of external stability checks are applied to the structure of elevated water tank including bearing capacity, sliding and overturning. Sliding and overturning checks are applied by calculating base shear and overturning moments from software. Stabilizing forces are calculated by multiplying superstructure weight including self-weight, water weight, live load and raft weight into the sliding coefficient μ=0.4.

Stabilizing moments are calculated by multiplying stabilizing force into the half width of the raft. The bearing capacity checks were applied to the EWT by considering pressure produce on soil due to the superstructure weight. These checks are only carried out in full condition (full from water) of the tank. **Table 3.4** shows the external stability checks of different EWTs. For the overturning check, stabilizing moment should be greater 2 times than overturning moment as per UBC-1997 code. For the sliding check, the resisting force should be greater 1.5 times than sliding force and for bearing pressure check, soil allowable bearing pressure q_e=1.5 TSF (3.3 ksi) should be greater than the pressure produce on soil due to the elevated water tank loads as per UBC-1997 code.

Table 3.3 shows external stability checks including overturning, sliding and bearing pressure. For model A_S, the resisting moment is 2.27 times greater than overturning moment, resisting force is than 1.69 times greater than sliding force and soil allowable bearing pressure q_e=3.3 ksi>2.23 ksi is greater than the pressure produce on soil by the weight of EWT. For model B_S, the resisting moment is 3.16 times greater than overturning moment, resisting force is than 2,51 times greater than sliding force and soil allowable bearing pressure q_e=3.3 ksi>161 ksi is greater than the pressure produce on soil by the weight of EWT. For model C_S, the resisting moment is 4.3 times greater than overturning moment, resisting force is than 3.21 times greater than sliding force and soil allowable bearing pressure q_e=3.3 ksi>1.33 ksi is greater than the pressure produce on soil by the weight of EWT. Values within the brackets are for the reduced thickness of EWTs. **Table 3.3** clearly shows that all external stability checks are ok for both EWTs (i.e. without and with reduced thickness of structural elements.

TABLE 3.3: External Stability Checks for Full Condition

Procedure	**Model**	**Overturning**	**Sliding**	**Bearing Pressure**
Static	A_S	2.27>2	1.69>1.5	2.23<qe
		(2.54>2)	(2.02>1.5	(1.93<qe)
Static	B_S	3.16>2	2.51>1.5	1.61<qe
		(3.22>2)	(2.99>1.5)	(1.58<qe)
Static	C_S	4.30>2	3.21>1.5	1.33<qe
		(4.40>2)	(3.83>1.5)	(1.24<qe)

Before the analysis of elevated water tank in SAP2000 software, views of the mashroom type elevated water tank are shown in **Fig. 3.4**.

Table 3.4: Number of Models to be Analyzed

Seismic Source Type as Per UBC-1997	**Seismic Magnitude (M_W)**	**Static Analysis**	**Pushover Analysis**		**Time History Nonlinear Analysis**			
					Confined		**Unconfined**	
			Confined	**Unconfined**	**Near**	**Far**	**Near**	**Far**
A	$M_w>7$	A_S	A_C	A_{UC}	A_{NLCn}	A_{NLCf}	A_{NLUCn}	A_{NLUCf}
B	$6.5<M_w<7$	B_S	B_C	B_{UC}	B_{NLCn}	B_{NLCf}	B_{NLUCn}	B_{NLUCf}
C	$M_w<6.5$	C_S	C_C	C_{UC}	C_{NLCn}	C_{NLCf}	C_{NLUCn}	C_{NLUCf}

Note: Values within brackets are for reduced thickness of the considered elevated water tanks.

2D view of the mashroom type elevated water tank is shown in **Fig. 3.4a** and 3D view of the mashroom type elevated water tank is shown in **Fig.3.4b**. In the x-direction, elevated water tank has windows and doors at both faces but in y-direction it does not have any door and window. It has two doors in the x-direction, one is at the ground level and the other one is at the mumty level (above of bowel top slab). In x-direction, it has 4 windows on both faces. Therefore, the stiffness of the elevated water tank in x-direction is less than y-direction.

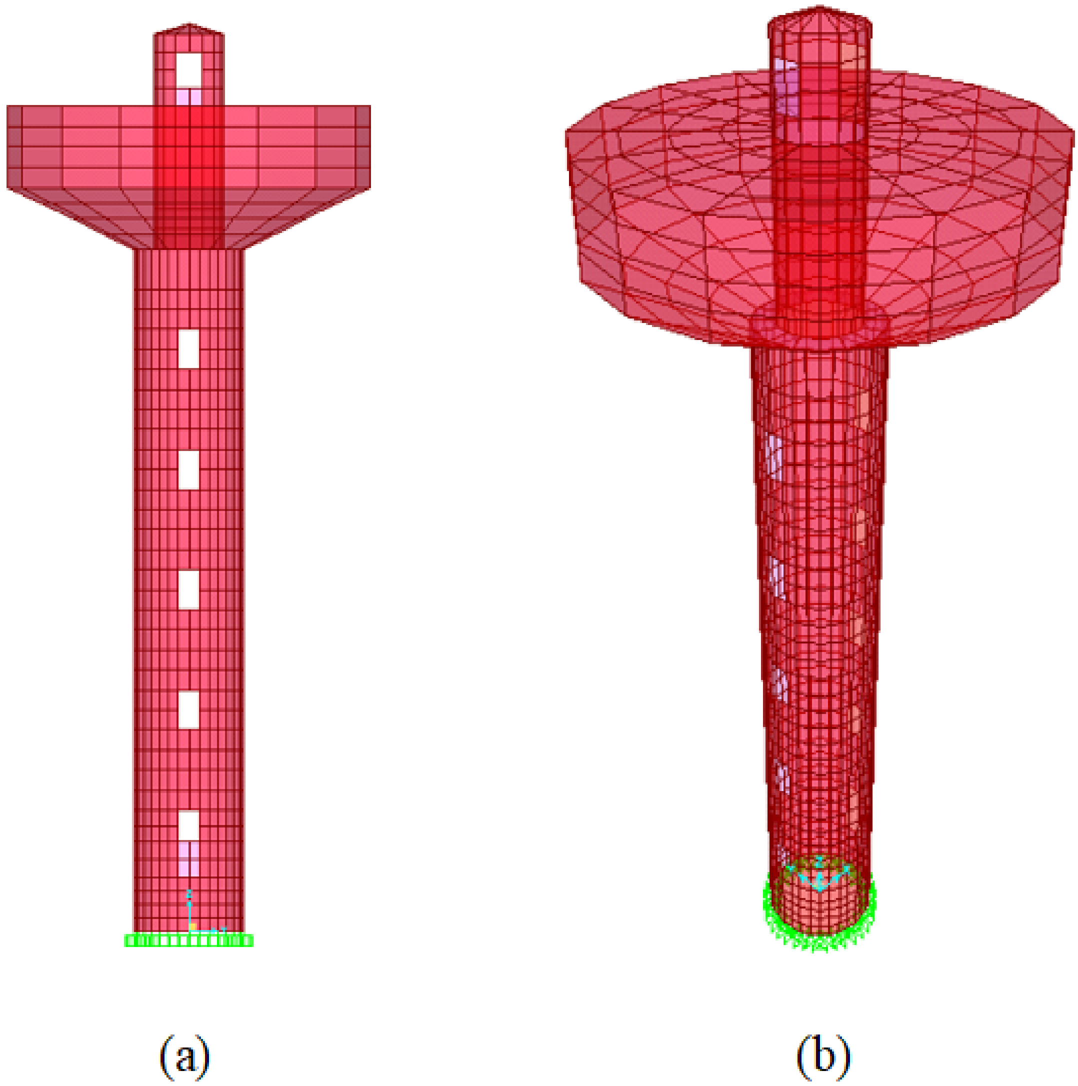

FIGURE 3.4: Modelling of Mashroom Type Elevated Water Tank in SAP2000; (a) 2D view; (b) 3D View

3.5 Parameters to be Analyzed

Today's advance tools i.e. finite element method for structural analysis allows detail modelling of the structures response. It is the well-known approach for the structures analysis by considering different characteristics of geometry and material nonlinearity. For the reliable simulation of the elevated water tank response and failure mechanism perform by pushover analysis. Pushover analysis evaluate structural condition during ground motion events with the incrementally horizontal loads application. The capacity of the elevated water tank is considered to be analyzed. Different capacity curves i.e. base shear verses displacement is obtained from the pushover analysis. The limit states of near collapse, significant damage and limited damage of the elevated water tank are obtained from the capacity curves of the elevated water tank. Time history nonlinear analysis is conducted to check the seismic displacement demand under near and far-fault earthquakes. Average exceedance ratio that refers to the structure cases which do not comply with a given performance level divided by total cases for the structure are determined.

3.6 Summary

Material properties and geometry of the elevated water tanks are described. Confined and unconfined concrete stress-strain relationship describe as per the method proposed by Kappos. External stability checks are conducted as per UBC-1997. All external stability checks are passed such as overturning check, sliding check and bearing pressure check. Equivalent static force procedure is performed as per UBC-1997 for soil profile type SD and seismic zone 4. Pushover analysis is carried out to check the capacity of elevated water tank models. Time history nonlinear analysis is conducted to check the displacement demand of EWTs.

Chapter 4

Results and Analysis

4.1 Background

Analysis of the elevated water tanks is done by using SAP2000 software. All governing moments are carefully considered and designed as per ACI-318 and ACI-371R codes. Capacity curves are obtained from the static nonlinear analysis for the confined and unconfined EWTs. Damage limit states are considered as per Eurocode 8. Near and far-fault ground motions are carefully considered for the time history nonlinear analysis and keep all the requirements of PEER database. Ground motion accelerations carefully matched with specified response spectrum as per UBC-1997 code for the soil profile type SD and seismic zone 4 by utilization of seismomatch software. Seismic displacement demands are illustrated and evaluated the performance levels. Exceedance ratios are determined by considering limited damage performance level, significant damage performance level and near collapse performance level. Average exceedance ratios are determined under near and far-fault earthquakes.

4.2 Equivalent Static Analysis

Models are analyzed according to equivalent static force procedure as per UBC-1997 code for the soil profile type SD and seismic zone 4.

Table 4.1: EWTs Longitudinal and Transverse Reinforcement Obtained from the Equivalent Static Analysis

Element	**Thickness**	$\mathbf{A}_C$		$\mathbf{B}_C$		$\mathbf{C}_C$		$\mathbf{A}_{UC}$		$\mathbf{B}_{UC}$		$\mathbf{C}_{UC}$	
Name	**(in)**	Lon.	Tran.	Lon.	Tran.	Lon.	Tran.	Lon.	Tran.	Lon.	Tran.	Lon.	Tran.
Shaft part-1	18	#7-4	#5-4	#6-4	#4-4	#5-4	#4-4	#7-4	#5-4	#6-4	#4-4	#5-4	#4-4
	(15)	(#8-6)	(#5-4)	(#7-6)	(#5-5)	(#6-6)	(#4-4)	(#8-6)	(#7-6)	(#5-5)	(#6-6)	(#4-4)	(#8-6)
Shaft part-2	18	#6-8	#5-4	#5-8	#4-4	#5-8	#4-4	#6-8	#4-6	#5-8	#4-6	#5-8	#4-8
	(15)	(#6-6)	(#5-4)	(#5-6)	(#5-5)	(#5-6)	(#4-4)	(#6-6)	(#5-6)	(#5-6)	(#4-5)	(#5-6)	(#4-6)
Shaft part-3	18	#7-8	#5-4	#6-6	#4-4	#5-8	#4-4	#7-8	#4-4	#6-6	#4-4	#5-8	#4-6
	(15)	(#7-6)	(#5-4)	(#6-6)	(#5-5)	(#5-6)	(#4-4)	(#7-6)	(#5-6)	(#7-6)	(#4-5)	(#6-6)	(#4-6)
Bowel base slab	36	#5-4	#5-4	#5-4	#5-4	#5-4	#5-4	#5-4	#5-4	#5-4	#5-4	#5-4	#5-4
	(30)	(#5-4)	(#5-4)	(#5-4)	(#5-4)	(#5-4)	(#5-4)	(#5-4)	(#5-4)	(#5-4)	(#5-4)	(#5-4)	(#5-4
Bowel Inclined slab	18	#5-4	#4-6	#5-4	#4-6	#5-4	#4-6	#5-4	#4-6	#5-4	#4-6	#5-4	#4-6
	(15)	(#5-4)	(#4-6)	(#5-4)	(#4-6)	(#5-4)	(#4-6)	(#5-4)	(#4-6)	(#5-4)	(#4-6)	(#5-4)	(#4-6
Bowel top slab	9	#4-8	#4-8	#4-8	#4-8	#4-8	#4-8	#4-8	#4-8	#4-8	#4-8	#4-8	#4-8
	(7)	(#4-8)	(#4-8)	(#4-8)	(#4-8)	(#4-8)	(#4-8)	(#4-8)	(#4-8)	(#4-8)	(#4-8)	(#4-8)	(#4-8

Continued Table: 4.1 EWTs Longitudinal and Transverse Reinforcement Obtained from the Equivalent Static Analysis

Element Name	**Thickness (in)**	$\mathbf{A}_C$ Lon.	Tran.	$\mathbf{B}_C$ Lon.	Tran.	$\mathbf{C}_C$ Lon.	Tran.	$\mathbf{A}_{UC}$ Lon.	Tran.	$\mathbf{B}_{UC}$ Lon.	Tran.	$\mathbf{C}_{UC}$ Lon.	Tran.
Bowel inner wall	12	#6-8	#4-8	#5-8	#4-8	#5-8	#4-8	#6-8	#4-8	#5-8	#4-8	#5-8	#4-8
	(10)	(#6-8)	(#4-8)	(#5-8)	(#4-8)	(#5-8)	(#4-8)	(#6-8)	(#5-8)	(#5-8)	(#4-8)	(#5-8)	(#4-8
Bowel outer wall	12	#4-8	#4-8	#4-8	#4-8	#4-8	#4-8	#4-	#4-8	#4-8	#4-8	#4-8	#4-8
	(10)	(#4-8)	(#4-8)	(#4-8)	(#4-8)	(#4-8)	(#4-8)	(#4-8)	(#4-8)	(#4-8)	(#4-8)	(#4-8)	(#4-8
Mumty slab	6	#4-8	#4-8	#4-8	#4-8	#4-8	#4-8	#4-	#4-8	#4-8	#4-8	#4-8	#4-8
	(6)	(#4-8)	(#4-8)	(#4-8)	(#4-8)	(#4-8)	(#4-8)	(#4-8)	(#4-8)	(#4-8)	(#4-8)	(#4-8)	(#4-8

Note 1: Reinforcements are in two layers without mumty, Lon.=longitudinal reinforcement and Tran.=transverse reinforcement.

Note 2: Values within bracket are for reduced thickness of the considered elevated water tanks.

Models are analyzed according equivalent static force procedure as per UBC-1997 code for the soil profile type SD and seismic zone 4. The analyzed models are designed as per ACI-318-08 code considering reinforcement minimum and maximum requirements. Basically, three EWTs are analyzed and designed mentioned in section 3.3 as per UBC-1997, ACI-371R and ACI-318-08 codes. Then, these three models are divided into six models by considering confinement of the concrete. The difference between confined and unconfined EWTs is in shaft transverse reinforcement.

In confined EWTs the spacing between transverse reinforcement are reduced in shaft part-2 and part-3 sections as compared to unconfined EWTs. Reinforcement of the bowel elements are kept same on confined A_C and unconfined A_{UC}, confined B_C and unconfined B_{UC} and confined C_C and unconfined C_{UC} EWTs. **Table 4.1** shows longitudinal and transverse reinforcement of the EWTs. Generally, reinforcement in bowel elements without bowel inner wall were same in all EWTs because the difference between governing moments were small. For the thickness reduced EWTs, reinforcements are shown in brackets, same as the procedure mentioned for the unreduced EWTs. Longitudinal and transverse reinforcement without mumty are in two layers.

4.3 Damage Limit States

Damage limit states are quantitative definition of performance levels by a suitable damage indicator capable of representing the seismic performance with appropriate damage thresholds. They should be determined in terms of quantitative measure of structural behavior such as displacement and deformation quantities. Such drift is effected by different factors including construction details, structural typology and boundary conditions as well as level of axial loading. Since, there are no values available in literature with observed ground motion damages for the case study of elevated water tanks for damage limit states. The investigated elevated water tanks capacity evaluation is performed using Eurocode 8. Three damage limit states that is near collapse (NC), significant damage (SD) and limited damage (LD) are considered as specified in the Eurocode 8 and several international guidelines such as [23, 39, 40].

In NC limit state, the structure is heavy damaged and has low residual lateral stiffness and strength, and vertical elements are able to withstand vertical loads. The structure is not able to survive another moderate intensity earthquake. Large permanent drift is present and the structure is near to collapse. NC damage limit state for the entire structure is the ultimate rotation θu capacity (elastic plus inelastic part) of the structure as per Eurocode 8. In SD limit state the structure has some residual lateral strength and stiffness, and vertical elements are able to withstand vertical loads. The structure is able to survive moderate intensity earthquakes. Moderate permanent drift is present and repairing of the structure is uneconomical. The drift capacity of SD limit state is equal to 75% of the NC damage state as per Eurocode 8 part 3. In LD limit state, the structure is lightly damaged. Structural elements are prevented from significant yielding and retaining their strength and stiffness. The damage should be economical and permanent drift is negligible, and structure does not need any repair measuring. For structure elements, limit state of LD is associated with the curve of force-displacement yield point. The LD limit state yield point drift θy is the rotation of the elastic branch. **Fig. 4.1** shows limit state rotations and bilinear force-deformation capacity of the EWT as per Eurocode 8.

4.4 Dynamic Characteristics

The modal analysis is performed by utilization of SAP2000 software and result of first 9 modes vibration are presented. The linear modal analysis results are incorporated in **Table 4.2** in terms of mass participating ratios and time periods. The first and second modes mass participation ratio are 80.7% and 00% for the x-direction, respectively while first and second modes mass participation ratio are 0.00% and 80.92% for the y-direction, respectively. Time period of the first and second modes are 0.99 and 0.92 sec are for both x and y, respectively. For vertical (z-direction), sixth mode is governing which mass participation ratio is

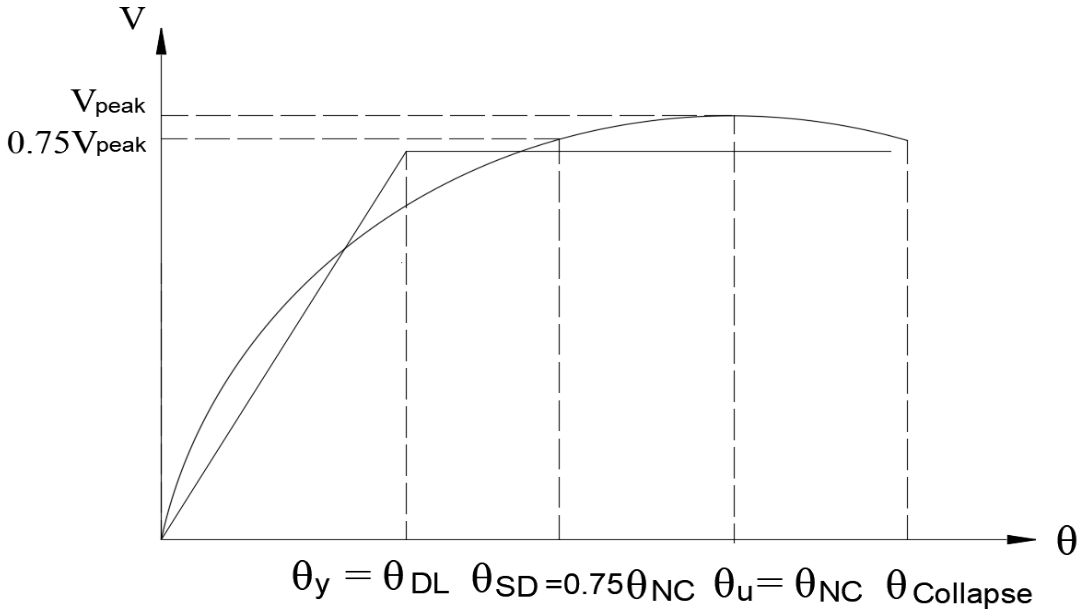

FIGURE 4.1: Limit State Rotations and Bilinear Force-Deformation Relationship of the EWT as Per Eurocode 8.

77.8%. For the reduced thickness elevated water tanks first mode mass participation ratios for the x-direction is 81.4%. The second mode mass participation ratio is 81.9% for y-direction. For the reduced thickness EWTs z-direction, sixth mode mass participation factor is 77.4%. First and second mode time period for the reduced thickness EWTs are 1.02 and 0.95 sec, respectively. For other modes time period and mass participation factors are mention in **Table 4.2**. All modes mass participation factors are same for AS, BS, and CS at both x and y directions. Reason behind the same modal mass participation ratios for AS, BS, and CS is the same geometry and water load. For the thickness reduced sections of EWTs, these values are presented within brackets in **Table 4.2**.

Note: Values within bracket are for reduced thickness of the considered elevated water tanks.

4.5 Strength and Deformation Capacity of Elevated Water Tanks

Pushover analysis (PA) is a nonlinear static analysis procedure which is used commonly in literature for the earthquake performance assessment [41]. In this

Table 4.2: Modal Mass Participation Ratio for First 9 Modes Vibration of the Design Template of A_S, B_S and C_S

Mode	**Period (sec)**	**M_x (%)**	**M_y (%)**	**M_z (%)**
1	0.98 (1.02)	80.7 (81.4)	0.00 (0.00)	0.00 (0.00)
2	0.92 (0.95)	0.00 (0.00)	80.9 (81.9)	0.00 (0.00)
3	0.27 (0.28)	0.00 (0.00)	0.00 (0.00)	0.00 (0.00)
4	0.15 (0.16)	8.01 (7.56)	0.00 (0.00)	0.00 (0.00)
5	0.14 (0.15)	0.00 (0.00)	7.87 (7.41)	0.00 (0.00)
6	0.11 (0.12)	0.00 (0.00)	0.00 (0.00)	77.8 (77.4)
7	0.06 (0.06)	0.00 (0.00)	5.00 (1.37)	0.00 (0.00)
8	0.05 (0.06)	4.56 (1.00)	0.00 (0.00)	0.00 (0.00)
9	0.04 (0.05)	0.00 (0.00)	0.00 (0.00)	0.47 (0.07)

procedure, important information about the structure like capacity and failure mechanism can be easily extracted using PA approach [42]. Pushover analysis is the combination of seismic response spectrum and nonlinear static analysis for the structural evaluation at inelastic demand. The pushover analysis consists of the application of gravity loads as a representative load pattern. Gravity loads are calculated by the combination of dead and live loads and are in place during lateral loading. Lateral forces are applied in the form of monotonic function in a stepwise. Dead loads are the self-weight of the EWT, superimposed dead load and water load. The structure response is considered through the capacity curve which basically given the relationship of the base shear force and displacement and since it is broadly accepted in practice.

The capacity curves of the elevated water tanks are determined by carried out pushover analysis in finite element SAP2000 software. For the scope of analysis, the pushover analysis has been carried out as per guidelines of ATC-40, FEMA-440 and FEMA-356. The target displacement has been calculated as per FEMA-273 and ATC-40 using equation-4.1. Distribution of the lateral loads are applied to the center of mass with the mode shapes considering the seismic weight. EWT seismic weight is calculated by the combination of 100% dead loads DLs and 25% live loads LLs (G+0.25LL). In this study, material nonlinearity is considered by using the nonlinear layers in finite element software. The geometry nonlinearity is not considered in the study. Elements of the EWT are modelled essential mechanical elastic and inelastic characteristic of reinforcement as concrete and steel separately [28]. Layers of the shell elements are based on the composites material mechanic principles in which coupled in plane bending-shear and coupled in plane/out plane bending nonlinear behavior of the RC elements. Reason the defining these properties by manual is the hinge properties disabled by default in the software. Because, when a single plastic hinge is occurred in EWT shaft, it could cause the collapse of the total structure before nonlinear resources of the rest of shaft remains fully utilized [28].

$$\Delta t = C_O C_1 C_2 C_3 S_a (\frac{T_e^2}{4\pi^2}) g \tag{4.1}$$

Where

Δt: target displacement.

C_o: the modification factor to relate spectral displacement and likely roof displacement.

C_1: modification factor of the maximum inelastic displacement.

C_2: the modification factor of represent the effect of hysteresis shape.

C_3: the modification factor of consideration P-delta effect.

S_a: the response spectrum acceleration in the direction under consideration of g.

T_e: effective fundamental time period.

g: the gravity acceleration.

Capacity curves of the elevated water tanks are shown in **Fig. 4.2**. Small displacement capacity is shown in x-direction compared to y-direction for each EWT. Because x-direction both sides have doors and windows, and the stiffness of the x-direction is less than compared to y-direction. Longitudinal reinforcement of the EWT A_C and A_{UC} are same, B_C and B_{UC} are same, and C_C and C_{UC} are same, but transverse reinforcements are different in shaft of the EWT. Strength and displacement capacities of EWTs A_C, B_C and C_C are higher compared to A_{UC}, B_{UC} and C_{UC}, respectively. Generally, strength and displacement capacities of the confined EWTs are higher compared to unconfined EWTs. Unconfined EWTs have less displacement capacities compared to confined EWTs. The reason behind strength and ductility increased significantly on confined EWTs is the higher transverse reinforcement ratio. This significant difference is clearly shown in **Fig. 4.2**. Transverse reinforcement prevent longitudinal reinforcement from bucking failure and keep the core concrete from spalling. Ultimate strength capacities of unreduced thickness EWTs of A_C, B_C and C_C, B_{UC} and C_{UC} are 1860, 1595, 1510, 1600, 1400, and 1360 kN at x-direction, respectively. however, theses capacities for unreduced thickness A_C, B_C and C_C, B_{UC} and C_{UC} are 2085, 1800, 1650, 1875, 1725, and 1590 kN at y-direction, respectively. For the reduced thickness EWTs the ultimate strength capacities of A_C, B_C and C_C, B_{UC} and C_{UC} at x-direction are 1615, 1475, 1320, 1375, 1250 and 1140 kN at x-direction, respectively. And, these capacities for reduced thickness A_C, B_C and C_C, B_{UC} and C_{UC} are 1805, 1670, 1550, 1655, 1620, and 1480 kN at y-direction, respectively. Thus, to increase the strength and ductility of the EWTs, arrange the proper ratio of the transverse reinforcement in shaft of the EWT.

Elevated water tanks damage limit states are calculated as per the outlined criteria of part-3 of the Eurocode-8. Seismic capacities of the EWTs are determined in **Table 4.3**. Limited damage is calculated by considering the roof yield displacement divided the total height of the EWT. Significant damage is calculated by considering 75% of the near collapse displacement divided the total height of the EWT. The near collapse has been calculated by the roof ultimate displacement divided total height of the EWT. **Table 4.3** clearly shows that displacement capacities of confined EWTs are higher compare to unconfined EWTs. The limited damage (LD) displacement capacities for thickness unreduced EWTs of AC and AUC are 0.58% and 0.26 at x-direction and 0.60% and 0.31% at y-direction, respectively. For BC and BUC EWTs the LD displacement capacities are 0.56 and 0.24% at x-direction and 0.57 and 0.25% for y-direction, respectively. For CC and CUC EWTs the LD displacement capacities are 0.57 and 0.22% at x-direction and 0.59 and 0.23% for y-direction, respectively. The displacement capacities of significant damage for unreduced thickness AC and AUC EWTs are 1.36% and 0.59 at x-direction and 1.89% and 0.91% at y-direction, respectively. For BC and BUC the SD displacement capacities are 1.16 and 0.61% at x-direction and 1.85 and 1.02% for y-direction, respectively. For thickness unreduced EWTs of CC and CUC the SD displacement capacities are 1.02 and 0.60% at x-direction and 1.62 and 1.09% for y-direction, respectively. Near collapse displacement capacities of the unreduced thickness of AC and AUC EWTs are 1.81% and 0.77% at x-direction and 2.52 and 1.21% at y-direction, respectively. For BC and BUC the NC displacement capacities are 1.61 and 0.82% at x-direction and 2.46 and 1.35% for y-direction, respectively. For EWTs of CC and CUC the NC displacement capacities are 1.36 and 0.80% at x-direction and 2.01 and 1.46% for y-direction, respectively. In thickness reduced EWTs, the displacement capacities of limited damage, significant damage and near collapse limit states are smaller than thickness unreduced EWTs are shown in the brackets of **Table 4.3**.

Table 4.3: Investigated Elevated Water Tanks Displacement Capacities Obtained from the Capacity Curves for the Considered Performance Levels

Model Design Identifier	**Direction**	**Limited Damage (LD)**		**Significant Damage (SD)**		**Near Collapse (NC)**	
		%Δroof /Helevated Tank					
A_C	x	0.58	(0.44)	1.36	(1.02)	1.81	(1.35)
	y	0.60	(0.45)	1.89	(1.56)	2.52	(2.06)
A_{UC}	x	0.26	(0.24)	0.59	(0.53)	0.77	(0.71)
	y	0.31	(0.23)	0.91	(0.86)	1.21	(1.15)
B_C	x	0.56	(0.46)	1.16	(0.98)	1.61	(1.31)
	y	0.57	(0.49)	1.85	(1.49)	2.46	(1.98)
B_{UC}	x	0.24	(0.23)	0.61	(0.48)	0.82	(0.64)
	y	0.25	(0.22)	1.02	(0.86)	1.35	(1.16)
C_C	x	0.57	(0.45)	1.02	(0.76)	1.36	(1.02)
	y	0.59	(0.47)	1.62	(1.41)	2.01	(1.88)
C_{UC}	x	0.22	(0.21)	0.60	(0.52)	0.80	(0.62)
	y	0.23	(0.19)	1.09	(0.81)	1.46	(1.08)

Note: Values within bracket are for reduced thickness of the considered elevated water tanks.

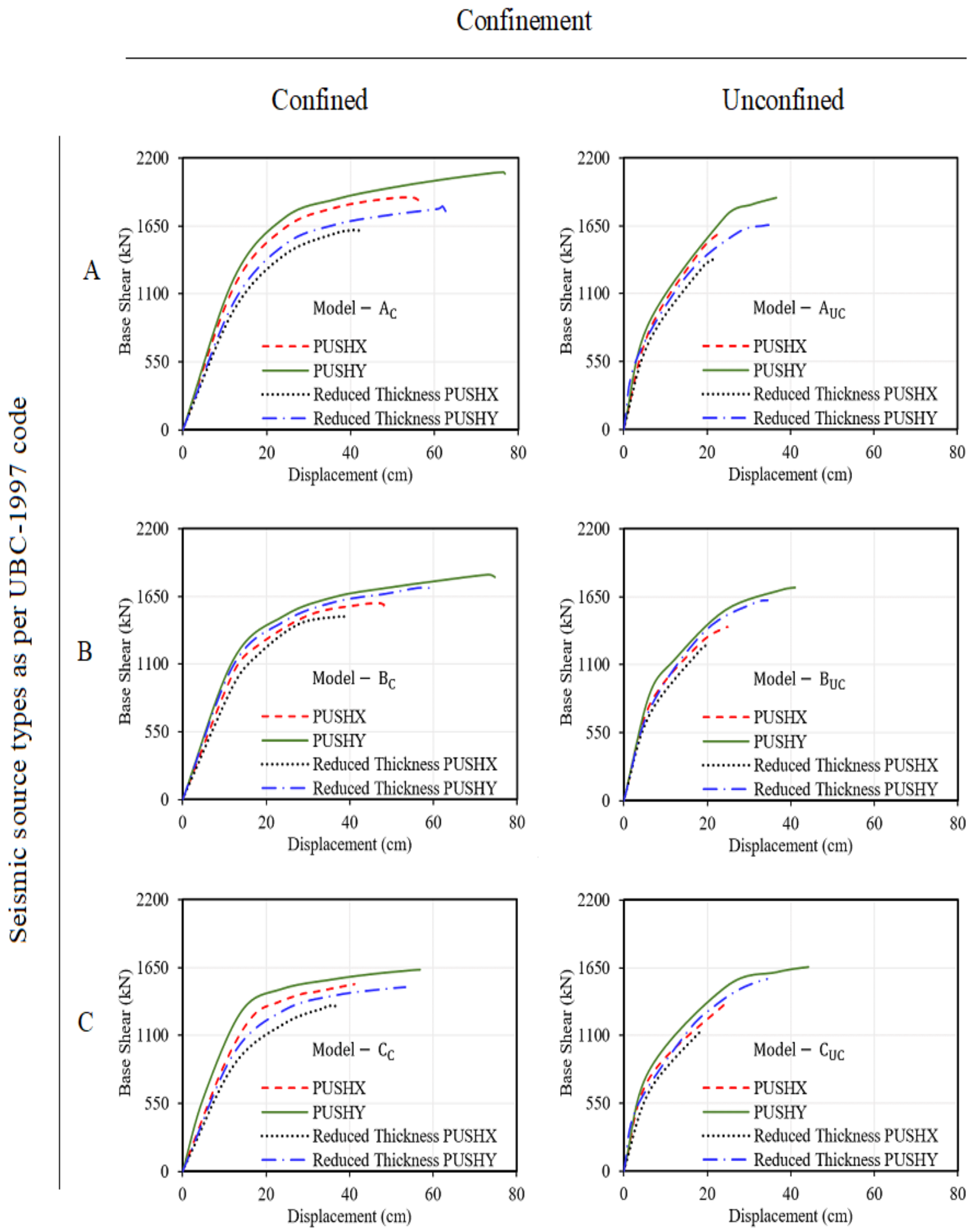

FIGURE 4.2: Capacity Curves for Confined and Unconfined EWTs Obtained from Pushover Analysis.

From the comparison of these results, it shows that confined EWTs have more strength and ductility compared to unconfined EWTs. Also, significant damage and near collapse capacities percentage ratios are higher for confined EWTs compared to unconfined EWTs. In thickness reduced EWTs, the displacement capacities of limited damage, significant damage and near collapse limit states are smaller than thickness unreduced EWTs are shown in the brackets **Table 4.3**.

4.6 Ground Motion Data and Estimation of Seismic Demand

The selection of ground motion is an important part of the time history non-linear analysis (THNLA). Total eight ground motion records are selected from Pacific Earthquake Engineering Research (PEER) database for THNLA with moment magnitude (M_w) between 6.19 and 7.9 including 4 near-fault and 4 far-fault records. Ground motion records are carefully selected from PEER database of NGW-WEST2 by completing the requirements of the PEER database. The near-fault ground motions are selected on the basis of their closest distance to the fault which is less than 10 km [8].

TABLE 4.4: Far and Near-Fault Ground Motion Records

Sr. No	PEER Record No	Location	Year	M_w	ClstD (km)	Vs30 (m/sec)	PGA (g)	Duration (sec)
Near								
1	2114	Denali, Alaska	2002	7.9	2.74	329.4	0.238	92.09
2	1605	Duzce, Turkey	1999	7.14	6.58	281.86	0.346	25.88
3	159	Imperial Valley-06	1979	6.53	0.65	242.05	0.472	28.44
4	1141	Dinar, Turkey	1995	6.4	3.36	219.75	0.141	27.96
Far								
5	1639	Manjil, Iran	1990	7.37	171.7	302.64	0.027	17.58
6	280	Trinidad	1980	7.2	76.26	311.75	0.028	19.68
7	426	Taiwan Smart 1(25)	1983	6.5	93.67	308.39	0.014	17.48
8	449	Morgan Hill	1984	6.19	39.08	288.62	0.045	28.36

Note: ClstD= Closest distance from the recording site to the ruptured fault area and Vs30= shear-wave velocity in the top 30 meters at the recording sites.

Acceleration time histories of origin al and matched ground motion records are plotted in **Figure. 4.3**.

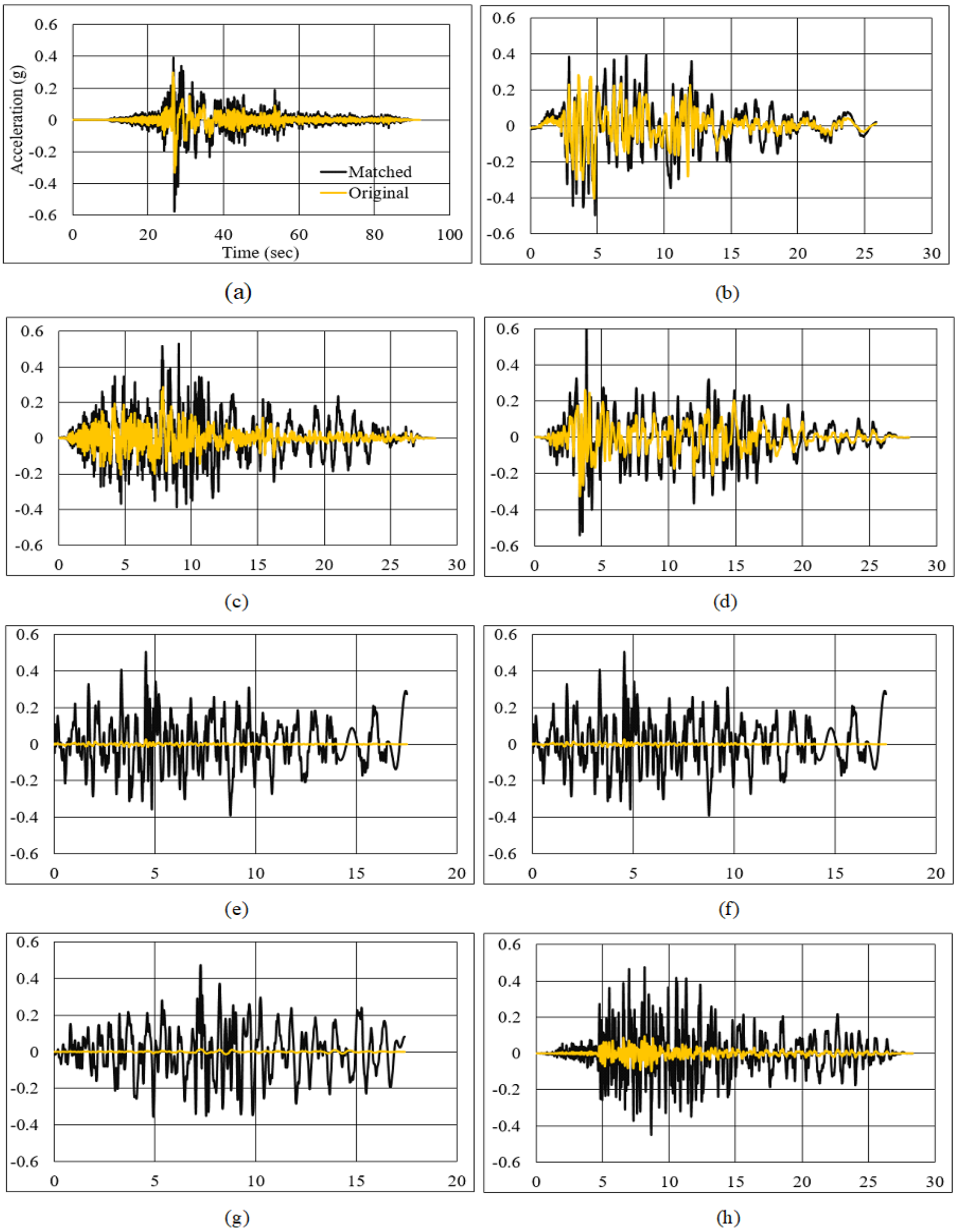

FIGURE 4.3: Acceleration of the Original and Matched Earthquakes in Horizontal Direction, (a) Denali, Alaska Earthquake, (b) Duzce, Turkey Earthquake, (c) Imperial Valley-06 Earthquake, (d) Dinar, Turkey Earthquake, (e) Manjil, Iran Earthquake, (f) Trinidad Earthquake, (g) Taiwan Smart 1(25) Earthquake, (h) Morgan Hill Earthquake.

Graphs shows different far and near fault ground motions in which y-direction shows acceleration and x-direction shows time.The original record is matched by utilization of seismomatch software for the response spectrum of soil profile type SD and seismic zone 4 as per UBC 1997. It shows that the difference between matched and original acceleration records are small due to near-fault records compared to far-fault records. Matched acceleration records of the Denali, Alaska earthquake, Duzce, Turkey earthquake, Imperial Valley-06 earthquake, Dinar, Turkey earthquake, Manjil, Iran earthquake, Trinidad earthquake, Taiwan Smart 1(25) earthquake, Morgan Hill earthquake are 0.578 g, 0.498 g, 0.528 g, 0.603 g, 0.507 g, 0.499 g, 0.476 g and 0.475 g, respectively. Where g is the acceleration due to gravity. The original accelerations of the ground motion records are listed in **Table 4.4**.

4.6.1 Equivalent Single Degree of Freedom Idealization of the Elevated Water Tank Response

The capacity curve of each elevated water tank is obtained from static pushover analysis with a billnear curve using the guidelines given in FEMA-440, FEMA-356, ATC-40 and Eurocode 8. Pushover/capacity and idelized capacity curves a typical example is shown in **Fig. 4.4**. The idealized capacity curve represent by yield and ultimate response points. ATC-40 and FEMA-356 provide guidelines for the capacity curve of equivalent single degree of freedom (SDOF) structures. Yield strength coefficient representation of equivalent SDOF structure is differ for both ATC-40 and FEMA-356 documents while yield displacement representation is same.

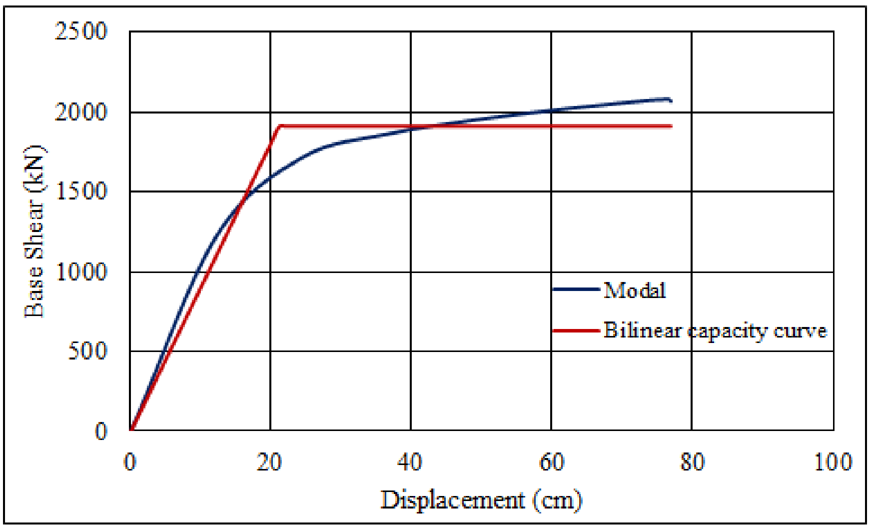

FIGURE 4.4: Idealization of a Typical Pushover Curve.

4.6.2 Time History Nonlinear Analysis and Performance Evaluation

Each investigated EWT is analyzed to determine the displacment demands subjected to earthquake records listed in **Table 4.4** with matching the records. In this method, nonlinear direct integration procedure is carried out by utilization Newmark average acceleration method with 5% constant damping ratio. Seismic perfomance of each elevated water tank is evaluated using **Table 4.3** ground motions considered in the study. **Table 4.5** shows the exceedance ratio of the EWTs by considering near collapse, significant damage and limited damage limit states. Exceedance ratio is the ratio, in which a case of EWTs do not comply a given performance level divided by the total cases of EWT. In performance evaluation, if the exceedance ratio is less than 0.5 then the performance level is considered to be satisfied and vice versa. The significant damage exceedance ratios of A_{NLC}, B_{NLC}, C_{NLC}, A_{NLUC}, B_{NLUC}, and C_{NLUC} unreduced EWTs are 0.25, 0.5, 0.5, 0.25, 0.5 and 0.25, respectively. For reduced thickness EWTs the exceedance ratios of A_{NLC}, B_{NLC}, C_{NLC}, A_{NLUC}, B_{NLUC}, and C_{NLUC} are 0.5, 0.75, 1.00, 1.00, 1.00 and 1.00, respectively. Every considered ground motion records in the study exceeds the limited damage performance level, because the average exceedance ratio is 1.00. Confined unreduced thickness EWTs satisfiy the significant damage performance level, because the exceedance ratio is 0.42. However, confined reduced thickness EWTs do not satify the significant damage performance level under near-fault ground motions, because the exceedance ratio is 0.75 which is more than 0.5. The average exceedance ratios for unconfined unreduced and reduced thickness EWTs are 0.83 and 1.00, respectively which are not statify in the performance evaluation. The average exceedance ratios for near collapse performance level for the confined unreduced and reduced thickness EWTs are 0.17 and 0.33 under near-fault ground motions, respectively which are statified in performance evaluation. The average exceedance ratio for unconfied reduced thickness EWTs 0.67 under near-fault ground motions which is not satisfied in the performance evaluation. However, unconfined reduced and unreduced EWTs satify the near collapse performacne level under far-fault ground motion.

Table 4.5: Average Exceedance Ratio of the Considered Performance Level for Far and Near-Fault Records of Different EWTs Design

Model	**Direction**	**Limited Damage (LD)**		**Significant Damage (SD)**		**Near Collapse (NC)**	
		Near-fault	Far-Fault	Near-fault	Far-Fault	Near-fault	Far-Fault
A_{NLC}	x	1.00 (1.00)	1.00 (1.00)	0.25 (0.50)	0.00 (0.25)	0.00 (0.25)	0.00 (0.00)
	y	1.00 (1.00)	1.00 (1.00)	0.00 (0.25)	0.00 (0.00)	0.00 (0.00)	0.00 (0.00)
A_{NLUC}	x	1.00 (1.00)	1.00 (1.00)	0.75 (1.00)	0.25 (0.50)	0.25 (0.50)	0.25 (0.25)
	y	1.00 (1.00)	1.00 (1.00)	0.25 (0.25)	0.25 (0.25)	0.25 (0.25)	0.00 (0.00)
B_{NLC}	x	1.00 (1.00)	1.00 (1.00)	0.50 (0.75)	0.00 (0.25)	0.00 (0.25)	0.00 (0.25)
	y	1.00 (1.00)	1.00 (1.00)	0.00 (0.25)	0.00 (0.25)	0.00 (0.25)	0.00 (0.00)
B_{NLUC}	x	1.00 (1.00)	1.00 (1.00)	1.00 (1.00)	0.50 (0.75)	0.75 (0.75)	0.25 (0.25)
	y	1.00 (1.00)	1.00 (1.00)	0.25 (0.25)	0.25 (0.25)	0.25 (0.25)	0.00 (0.00)
C_{NLC}	x	1.00 (1.00)	1.00 (1.00)	1.00 (1.00)	0.50 (0.75)	0.50 (0.50)	0.00 (0.25)
	y	1.00 (1.00)	1.00 (1.00)	0.25 (0.25)	0.25 (0.25)	0.00 (0.25)	0.00 (0.00)
C_{NLUC}	x	1.00 (1.00)	1.00 (1.00)	0.75 (1.00)	0.25 (0.50)	0.50 (0.75)	0.25 (0.25)
	y	1.00 (1.00)	1.00 (1.00)	0.25 (0.50)	0.25 (0.25)	0.00 (0.25)	0.00 (0.25)
Averages							
Confined	x	1.00 (1.00)	1.00 (1.00)	0.42 (0.75)	0.17 (0.42)	0.17 (0.33)	0.00 (0.17)
	y	1.00 (1.00)	1.00 (1.00)	0.08 (0.25)	0.08 (0.08)	0.00 (0.17)	0.00 (0.00)
Unconfined	x	1.00 (1.00)	1.00 (1.00)	0.83 (1.00)	0.25 (0.33)	0.50 (0.67)	0.25 (0.25)
	y	1.00 (1.00)	1.00 (1.00)	0.25 (0.33)	0.08 (0.25)	0.08 (0.25)	0.00 (0.08)

Note: Values within bracket are for reduced thickness of the considered elevated water tanks.

4.7 Summary

Capacity curves are obtained as per static nonlinear analysis for confined and unconfined EWTs. Confined EWTs have higher strength and displacement capacity than unconfined EWTs. The yield and ultimate strength and displacement capacities of confined EWTs were much higher than unconfined EWTs. Limited damage, significant damage and near collapse performance levels were evaluate as per Eurocode 8 under near and far-fault earthquakes. Near fault earthquakes significant damaged the unconfined EWTs than far-fault earthquakes. Near and far-fault earthquakes limited damage confined EWTs. Near collapse limit state was observed on unconfined EWTs under near-fault earthquakes. The exceedance ratio showed that unconfined EWTs did not satisfy the significant damage performance level under near fault earthquakes while confined EWTs satisfied that ratio. All confined and unconfined thickness unreduced EWTs satisfied the near collapse performance level under considered earthquakes. however, thickness reduced unconfined EWTs did not satisfied it.

Chapter 5

Discussion

5.1 Background

In this chapter, the exceedance ratios of near collapse, significant damaged and limited damaged performance levels are discussed for the confined and unconfined EWTs under both far and near-fault records. The average exceedance ratio of the performance level for confined and unconfined EWTs are compare and the effects of far and near-fault earthquakes on EWTs are discussed. Guidelines for the practical designers are discussed when they design the EWTs and practical implement in the field.

5.2 Comparison of Current Study with Previous Studies

The observed damages on elevated water tanks during past earthquakes were noticed in several studies [1, 2, 5, 7]. It is well-known that major number of EWTs were damaged at different levels as mentioned in the section 2.2 during recent earthquakes. Literature studies showed that unconfined concrete structural elements were more damaged during earthquakes compared to confined concrete structural elements. The reason for such damages is insufficient ductility to prevent permanent drift. Reinforced concrete shaft confined by transverse reinforcement significantly improved ductility and strength of the EWT. The effects of near-fault earthquakes records were more than compared to far-fault records.

5.3 Guidelines for Practical Designers

Average exceedance ratios of the EWTs are summarized in **Table 4.5** for far and near-fault records of the considered performance levels. If the average exceedance ratio is smaller than 0.50, then the performance level is considered to be satisfied in the performance evaluation. As it has seen in **Table 4.5**, limited damage performance level exceeded in every EWT. The average exceedance ratio of LD performance level is 1 for far and near-fault earthquakes. Significant damage performance level (PL) exceeded in majority unconfined EWTs due to near-fault earthquakes.

Table 4.5 clearly shows that near-fault earthquakes effects on the seismic response of elevated water tanks are notable on the LD and SD performance levels. More than half of considered EWTs especially unconfined EWTs were critical for significant damage performance level suggesting immediate planning and necessary safeguard need to become in action. **Table 5.1** shows the performance evaluation of the EWTs. Confined and unconfined reduced thickness EWTs do not satisfied the significant damage performance level as per Eurocode 8. However, unreduced thickness confined EWTs satisfied the significant damage performance level. The near collapse performance level is satisfied for all considered EWTs as it has seen in table 5.1. The limited damage performance level is not satisfied for all the considered EWTs. Limited damage performance level is acceptable for the elevated water tanks because it does not need any repairing, structure is in elastic range and it keeps functions. Significant damage performance level is not acceptable for the elevated water tanks because permanent drift is occurring, repairing is uneconomical and structure does not keep functions. Thus, elevated water tanks should be designed to satisfy the significant damage performance level. Confinement of the concrete and sufficient thickness of the shaft should be considered in the design of EWTs to reduce the effects of significant damage.

TABLE 5.1: Performance Evaluation of the Considered EWTs as Per Eurocode 8

Performance Evaluation	**Unconfined Shaft Thickness**		**Confined Shaft Thickness**	
Satisfaction	**15 (in)**	**18 (in)**	**15 (in)**	**18 (in)**
LD/SD/NC	NC	NC	NC	SD

Note: LD=limited damage, SD=significant damage and NC=near collapse

Fig. 5.1 shows performance based seismic design procedure. First of all, select the seismic performance objects that is earthquake design level, performance level and design criteria.

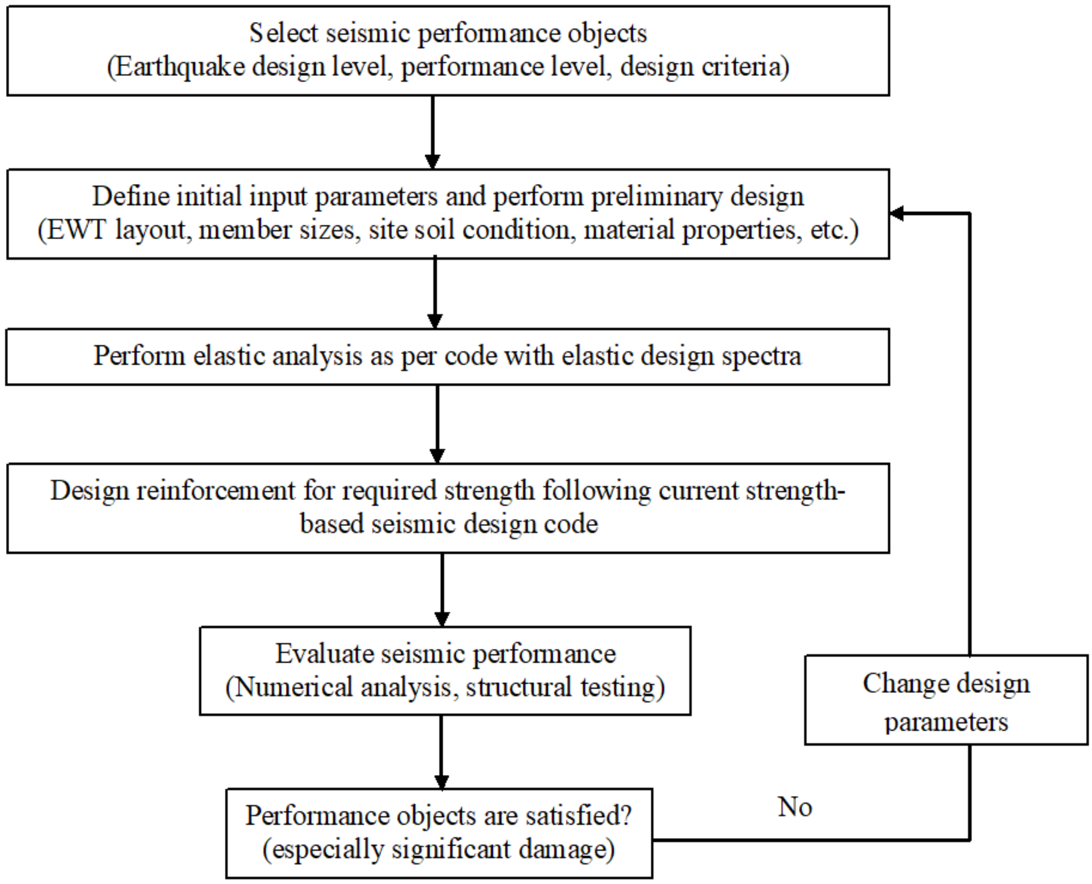

FIGURE 5.1: Flowchart of the Performance Based Seismic Design Procedure

After that define initial input parameters such as EWT layout, member sizes, site soil condition, material properties and etc. and perform preliminary design. Perform elastic analysis as per code with elastic design spectra and design the EWT for the required strength as per code. Evaluate seismic performance of the designed EWT. If the performance objects are not satisfied, especially significant damage performance level then start the procedure again form the initial input parameters.

5.4 Summary

In this chapter outcome of research work with practical needs are explained. Reinforced concrete shaft confined by transverse reinforcement improved both strength and ductility of the EWTs. The performance level is considered to be satisfied in the performance evaluation if the average exceedance ratio is smaller than 0.50. The average exceedance ratio of limited damage performance level is not satisfied for both confined and unconfined EWTs under far and near-fault records. However, significant damage performance level is satisfied for only confined EWTs. The near collapse performance level is satisfied for both confined and unconfined EWTs under near and far-fault records. EWTs should be satisfied the significant damage performance level because they are essential structures in any society and must remain functional after earthquake.

Chapter 6

Conclusion and Future Work

6.1 Conclusion

Pakistan is located in the seismic active area and there are located a large number of disastrous earthquakes in the history of Pakistan. In this study, a comparative assessment of the near and far-fault earthquakes to evaluate the seismic performance of EWTs. Three dimensional EWT models were simulated, and material nonlinearity was considered by utilizing finite element SAP2000 software. Static nonlinear analysis was accomplished to check the capacity of EWTs, and time history nonlinear analysis was performed to evaluate the seismic performance levels of the considered EWTs. The outcomes of the study are presented as follows.

- Equivalent static force procedure showed that governing moments, base shear forces and displacement were higher for seismic source type A, B and C, respectively.
- First and second mode of the mass participation ratios were governing for x and y-direction of the EWTs, respectively which was more than 80%.
- Limited damage is considered when displacement demand is greater than yield capacity and near collapse is considered when this demand is reach to ultimate displacement. Significant damage is equal to 75% of the near collapse as per Eurocode 8.

- Confined EWTs resulted in higher strength and displacement capacities compared to unconfined EWTs.

- The difference between original and matched acceleration is smaller for the near-fault ground motions while for far-fault ground motions, it is higher, because near-fault ground motions have higher PGA compare to far-fault ground motions. And, higher displacement demands resulted under near fault ground motion records compared to far-fault records.

  - Bilinear curve of the elevated water tanks represents by the yield and ultimate response points.

  - Confined EWTs satisfy the significant damage performance level un der near-fault ground motions, but unconfined EWTs do not satisfy it. Both thickness reduced and unreduced confined EWTs satisfy the near collapse performance level under near and far-fault ground motions. However, unconfined reduced EWTs do not satisfy the near collapse performance level, but unconfined unreduced thickness EWTs satisfy it under near-fault ground motions.

- Result showed that average exceedance ratio of the limited damage performance level was not satisfied for the studied EWTs. However, majority of confined EWTs satisfied the significant damage performance level, but unconfined EWTs did not satisfy it.

Confinement of the concrete by transvers reinforcement and sufficient thickness of the shaft shows effectiveness on the enhancement of strength and displacement capacities of EWTs. It is recommended to consider the transverse reinforcement throughout the height of shaft which is obtained at the base of shaft.

6.2 Future Work

Following recommendations are drawn for future works:

- Consider both material and geometry nonlinearity on the analysis of confined and unconfined EWTs under far and near-fault earthquakes.

- Check cracks due to far and near-fault earthquakes on confined and unconfined EWTs shaft.

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