APPLICATION OF TUNED LIQUID DAMPER FOR CONTROLLING STRUCTURAL VIBRATION

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF
THE REQUIREMENTS FOR THE DEGREE OF

Master of Technology

in

Structural Engineering

By

BHARADWAJ NANDA

DEPARTMENT OF CIVIL ENGINEERING
NATIONAL INSTITUTE OF TECHNOLOGY
ROURKELA-769008
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Under the guidance of

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ROURKELA

CERTIFICATE

This is to certify that the thesis entitled, ***"APPLICATION OF TUNED LIQUID DAMPER FOR CONTROLLING STRUCTURAL VIBRATION"*** *submitted by* ***Bharadwaj Nanda*** *in partial fulfillment of the requirements for the award of Master of Technology Degree in Civil Engineering with specialization in "****Structural Engineering****" at National Institute of Technology, Rourkela is an authentic work carried out by him under my supervision and guidance. To the best of my knowledge, the matter embodied in this Project review report has not been submitted to any other university/ institute for award of any Degree or Diploma.*

Date: *25th MAY 2010*

(Prof. K.C. Biswal)
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Dedicated to my parents

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ABSTRACT:

Current trends in construction industry demands taller and lighter structures, which are also more flexible and having quite low damping value. This increases failure possibilities and also, problems from serviceability point of view. Several techniques are available today to minimize the vibration of the structure, out of which concept of using of TLD is a newer one. This study was made to study the effectiveness of using TLD for controlling vibration of structure. A numerical algorithm was developed to investigate the response of the frame model, fitted with a TLD. A linear TLD model was considered. A total of six loading conditions were applied at the base of the structure. First one was a sinusoidal loading corresponding to the resonance condition with the fundamental frequency of the structure, second one was corresponding to compatible time history as per spectra of IS-1894 (Part -1):2002 for 5% damping at rocky soil and rest four were corresponding to time histories of past earthquake such as 1940 El Centro Earthquake record (PGA = 0.313g), 1994 North Ridge Loading (PGA = 1.78g), 1971 Sanfernando Earthquake (PGA = 1.23g), 1989 Loma Prieta Earthquake (PGA = 0.59g). A ten storey and two bay structure was considered for the study. The effectiveness of the TLD was calculated in terms of amplitude of displacements at top storey of the structure.

From the study it was found that, TLD can effectively used to control the vibration of the structure. TLD was more effective when it is placed at the top storey of the structure. Only TLD which were properly tuned to natural frequency of structure was more effective in controlling the vibration. The damping effect of TLD is sharply decreases with mistuning of TLD.

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CHAPTER - 1 (INTRODUCTION & BACKGROUND)

1.1 INTRODUCTION:

Now-a-days there is an increasing trends to construct tall structures, to minimize the increasing space problems in urban areas. These structures are often made relatively light & comparatively flexible, possessing quite low damping, thus making the structure more vibration prone. Besides increasing various failure possibilities, it may damage cladding and partitions and can cause problems from service point of view. Therefore, to ensure functional performance of tall buildings, it is important to keep the frequency of objectionable motion level bellow threshold. Various possibilities are available to achieve this goal [1, 2] are presented below:

Means	Type	Methods	Remarks
Aerodynamic design	Passive	Improving aerodynamic properties to reduce wind force coefficient	Chamfered corners and Openings
Structural Design	Passive	Increasing Building mass to reduce air/building mass ratio	Increased material cost
		Increasing Stiffness or natural frequency to reduce non-dimensional wind speed	Bracing walls, Thick members
Auxiliary damping device	Passive	Addition of materials with energy dissipative properties, Increasing building damping ratio	SD, SJD, LD, FD, VED, VD, OD
		Adding auxiliary mass system to increase level of damping	TMD,TLD
	Active	Generating control force using inertia effects to minimize response	AMD, AGS
		Generating aerodynamic control force to reduce wing force coefficient or minimize response	Rotor jet, Aerodynamic Appendages
		Changing stiffness to avoid resonance	AVS

Means	Type	Methods	Remarks
	Hybrid	Employs a combination of both active and passive control systems in order to alleviate some of the restrictions and limitations that exists when each system is acting alone	Hybrid Base Isolation, Hybrid Mass Damper
	Semi active	Cannot inject mechanical energy into the controlled structural system (i.e., including the structure and the control device), but has properties which can be controlled in real time to optimally reduce the responses of the system.	VOD, VFD, Controllable TLD, Controllable Fluid Damper (employing Electro Rheological & Magneto Rheological fluid), Semi active Impact Dampers

SD: Steel dampers, SJD: Steel Joint dampers, LD: Lead dampers, FD: Friction Dampers, VED: Visco-Elastic Dampers, VD: Viscous Dampers, OD: Oil Dampers, TMD: Tuned Mass Dampers, TLD: Tuned Liquid Dampers, AMD: Active Mass Damper, AGS: Active Gyro Stabilizer, AVS: Active Variable Stiffness, VOD: Variable Orifice Damper, VFD: Variable Friction Damper

Along with some unique advantages, all of these techniques have some of their own restrictions & disadvantages. However, the use of Tuned Liquid Dampers (TLDs), comprising both Tuned Sloshing Dampers (TSDs) and Tuned Liquid Column Damper (TLCDs), are gaining wide acceptance as a suitable method of structural control.

1.2 HISTORY:

Since 1950s dampers utilizing liquid is being used in anti-rolling tanks for stabilizing marine vessels against rocking and rolling motions. In 1960s, the same concept is used in Nutation Dampers used to control wobbling motion of a satellite in space. However, the idea of applying TLDs to reduce structural vibration in civil engineering structures began in mid 1980s, by Bauer [3], who proposed the use of a rectangular container completely filled with two immiscible liquids to reduce structural response to a dynamic loading. Modi & Welt [4], Fujii et al. [5], Kareem [6], Sun et al. [7], and Wakahara et al. [8] were also among the first to suggest the use of dampers utilizing liquid motion for civil engineering structures. The principles of operation of all of these dampers were based on liquid sloshing, for which these are sometimes referred as Tuned Sloshing Damper (TSDs).

Several other types of liquid dampers are also proposed during last two decades, out of which Tuned Liquid Column Damper (TLCDs) [9,10] which suppresses the wind induced motion by dissipating the energy through the motion of liquid mass in a tube like container fitted with orifice, is well known.

1.3 CLASSIFICATION:

Fig 1.1 bellow shows a schematic diagram of Tuned Liquid Damper family.

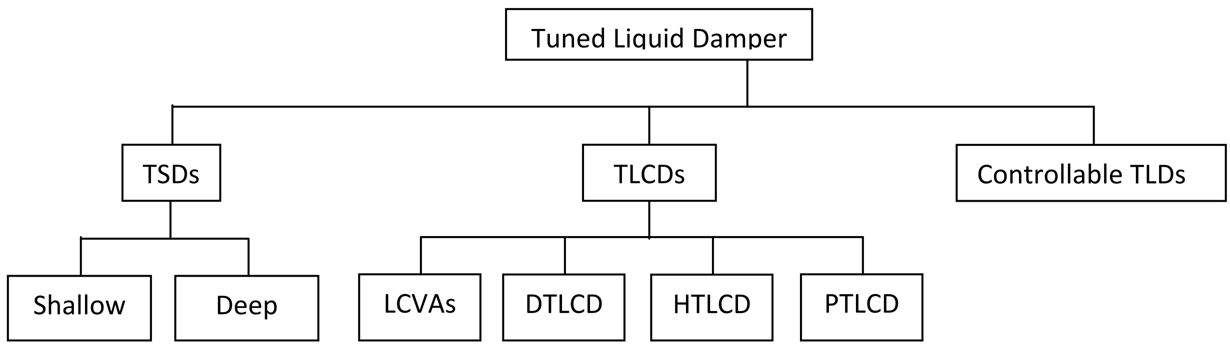


Fig. 1.1 (*Schematic of Tuned Liquid Damper family*)

TSD: Tuned Sloshing Damper, TLCD: Tuned Liquid Column Damper, LCVA: Liquid Column Vibration Absorbers, DTLCD: Double Tuned Liquid Column Damper, HTLCD: Hybrid Tuned Liquid Column Damper, PTLCD: Pressurized Tuned Liquid Column Damper, ER: Electro Rheological, & MR: Magneto Rheological.

1.3.1 Tuned Sloshing Damper:

Tuned Sloshing Dampers (TSDs) [4-8] are generally rectangular type or circular type and are installed at the highest floor according to building type and the objective for controlling the vibration. A TSD can be classified as shallow water type or deep water type depending on height of water in the tank. This classification of the TSDs is based on shallow water wave theory [11]. If the height of water 'h' against the length of the water tank in the direction of excitation 'L' (or diameter 'D' in case of circular tank) is less than 0.15 it can be classified as shallow water type else as deep water type if is more than 0.15. Fig. 1.2 shows the schematic of a TSD. The depth of the liquid in a container could be deep or shallow, depending on the

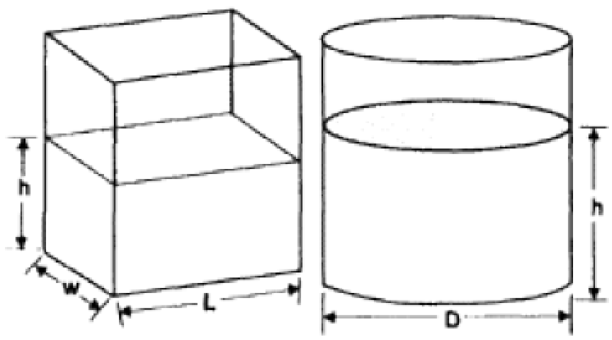


Fig. 1.2 (*Tuned Liquid Damper Dimensions*)

natural frequencies of the structure under control. Shallow water type has a large damping effect for a small scale of externally excited vibration, but it is very difficult to analyze the system for a large scale of externally excited vibration as sloshing of water in a tank exhibits nonlinear behavior. In case of deep water type, the sloshing exhibits linear behavior for a large scale of externally excited force [12].

When frequency of tank motion is close to one of the natural frequencies of tank fluid, large sloshing amplitudes can be expected. If both frequencies are reasonably close to each other, resonances will occur. Generally tuning the fundamental sloshing frequency of the TLD to the structures natural frequency causes a large amount of sloshing & wave breaking at the resonant frequencies of the combined TLD-Structure system, this dissipate a significant amount of energy [13].

As a passive energy dissipation device TLD presents several advantages over other damping systems such as (i) Low installation and RMO (Running, Maintenance and Operation) cost, (ii) Fewer mechanical problem as no moving part is present, (iii) Easy to install in new as well as in existing buildings as it does not depends on installed place and location, (iv) It can be applied to control a different vibration type of multi-degree of freedom system which has a different frequency for each other (v) Applicable to temporary use (vi) Non restriction to unidirectional vibration (vii) Natural frequency of TLD can be controlled by adjusting the depth of liquid and container dimensions, and (vii) Water present in the damper can be used for fire-fighting purpose.

Along with the above mentioned advantages, there are some drawbacks too associated with TLD system. The main drawback of a TLD system that, all the water mass does not participate in counteracting the structural motion [14]. This results the addition of extra weight without getting the any benefit. Again low density of water makes the damper bulky, and hence increase the space required housing it. As is the case for Tuned Mass Damper, there exists an optimal damping factor for TLDs. Since usually plain water is used as working fluid, it gives a lower damping ratio compared to the optimal value.

In order to overcome the drawbacks and to achieve the optimal damping ratio, several methods are proposed such as (i) Installing the TLD at proper position [15] (ii) Wave breaking in shallow water TLDs [7], (iii) Addition of floating beads as surface contaminants [16], (iv) Using submerged nets and screens [17-20], (v) Using Slopped bottoms for TLD [21-23], (vi) Enhancement of bottom roughness by using wedge shaped bottom with steps and with holes [23], (vii) Using a conical TLD [24], and (viii) Inserting poles [25].

1.3.2 Tuned Liquid Column Damper:

Unlike Tuned Sloshing Damper, which depends on liquid sloshing for dampening the structural vibration, Tuned Liquid Column Dampers (TLCDs), dissipates structural vibration by combined action involving the motion of the liquid mass in the tube, where the restoring force is due to the gravity acting upon the liquid and the damping effect as a result of loss of hydraulic pressure due to the orifice (s) installed inside the container [9]. Fig 1.3 shows the schematic of a TLCD.

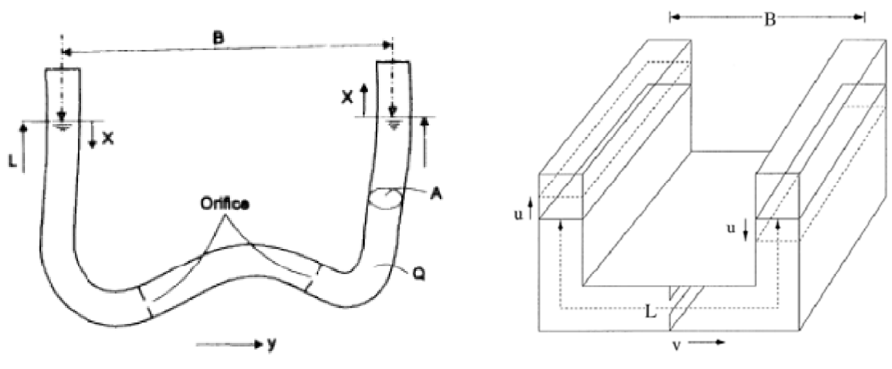


Fig. 1.3 (*Tuned Liquid Column Damper Dimensions*)

As a damper TLCD system offers few advantages over other damping devices, including TSDs, such as: (i) TLCD can take any arbitrary shape, for which it can be fitted to an existing structure easily, (ii) unlike its counterpart, TSDs, the mechanism of TLCD is well understood, and hence a accurate mathematical model, which quantitatively defines the dynamics of TLCD, can be formulated, (iii) We can control the damping capacity of TLCD through controlling orifice opening. This allows us to actively control the damping in TLCD system, and (iv) We can tune the frequency of a TLCD by adjusting the liquid column in the tube.

More recently Liquid Column Vibration Absorber (LCVA), a variation of TLCD, has been proposed [26-29]. The major difference between a TLCD and a LCVA is that, the cross section of the LCVA is not uniform. Since it has different dimensions for vertical and horizontal portions of container, it has benefits of easy tuning and wide range of natural frequency, as the natural frequency of the LCVA is determined not only by the length of the liquid column but also the geometric configuration.

One of the major disadvantages of TLCD and LCVA system is their unidirectional nature of action, and hence they can be applied effectively to the structure, which oscillates in only one predominant plane, but not to the structure that oscillates in bidirectional plane. To overcome this difficulty, a system has been proposed, named Double Tuned Liquid Column Damper (DTLCD), which consists of two TLCD in orthogonal directions [30]. Fig 1.4 shows the schematic of a DTLCD.

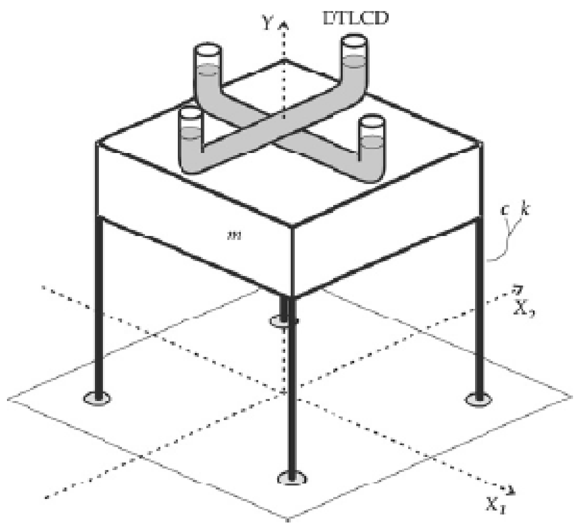


Fig. 1.4 (*a Double Tuned Liquid Column Damper*)

A Hybrid fluid dynamic system, named Hybrid Tuned Liquid Column Damper (HTLCD), has also been provided to overcome the above difficulty [31]. This system consists of a unidirectional TLCD fixed on the surface of a rotatable circular platform whose motion is controlled by an electrical-mechanical system. This hybrid system is passive in generation of control force to attenuate the displacement amplitudes, where as active in searching the right direction. Fig 1.5 shows the schematic of a HTLCD.

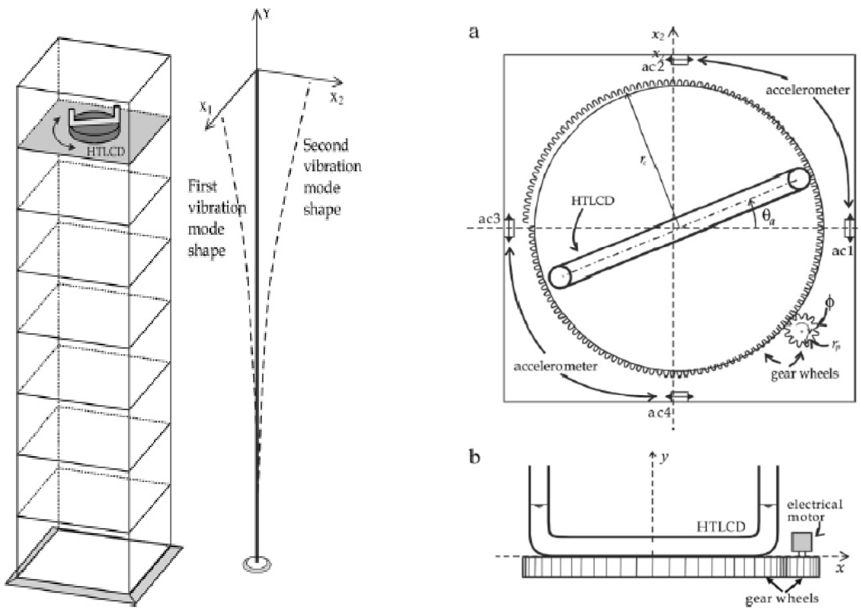


Fig 1.5 (*a Hybrid Tuned Liquid Column Damper*)

By implementing a static pressure inside two sealed air chambers at two ends of a TLCD, a new kind of TLCD is formed, whose frequency can be adjusted by both the length of its liquid column and the pressure inside its two air chambers. This is called Pressurized tuned liquid Column Damper (PTLCD) [32]. Fig 1.6 shows the schematic of a HTLCD.

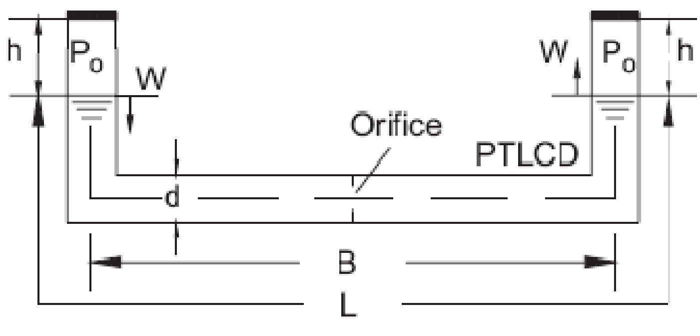


Fig 1.6 (*a Pressurized Tuned Liquid Column Damper*)

1.3.3 Controllable Tuned Liquid Damper:

Being a passive control device, TLDs are generally tuned to a particular frequency (1st natural frequency of structure), and therefore it is effective only if the frequency of forcing function is close to that tuned frequency. But in reality, the forces that act on the structure are often spread over a band of frequencies. This reduces the effectiveness of the damper. In order to improve the effectiveness of damping, against a multi-frequency excitation force, some active or semi-active control devices are proposed by various researchers.

Fig 1.7 provides a schematic diagram for a structural control problem. In a structural control problem (Active or Semi-active), the excitation force and the response of the structure to the excitation force are measured by the sensors, installed at key locations of the structure. Then the measured force and response are sent to a control computer, which processes them according to a control algorithm, and sends an appropriate signal to the actuators. The actuator then modifies the dynamic characteristics of the damper, to apply the inertial control forces to the structure in the desired manner.

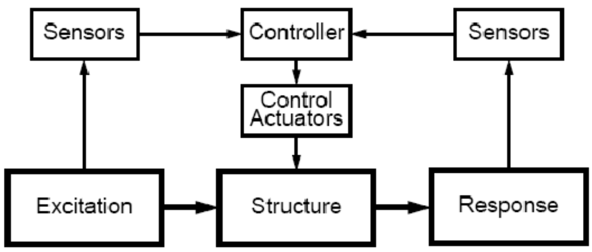


Fig 1.7 (*schematic diagram for a structural control problem*)

Several means for actively controlled Tuned Liquid Dampers are proposed, such as: (i) controlling the angle of baffles, in case of a TSD, regulates the effective length of the damper, which in turn adjusts the resonance frequency of the TLD [33, 34]. (ii) Installing one or more propellers driven by a servo-motor controlled by computer inside the horizontal section of TLCD. Both the fluid acceleration and the thrust generated by propeller acts simultaneously to increase vibration control ability significantly [35]. (iv) Balendra et al. [36] proposed an actively controlled TLCD (Fig 1.8) which is fixed on a movable platform at the top of the tower. The movement of the platform is controlled by a controlled force. A servo-actuator is used to generate the control force based on the feedback from the sensor attached to the top of the tower.

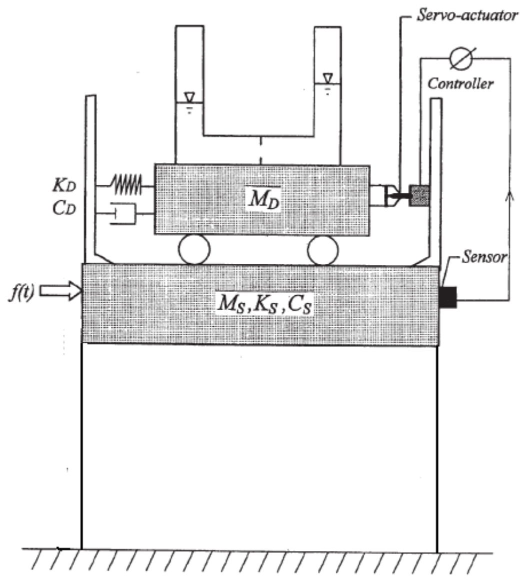


Fig 1.8 (*schematic diagram of the Active TLCD proposed by Balendra et al.*)

The Tuned liquid dampers can be controlled by following means, (i) Changing the orifice opening in case of a TLCD system, to control head loss coefficient [37, 38]. (ii) Using Electro-Rheological Fluid (ER-Fluid) [39] or Magneto-Rheological fluid (MR-Fluid) [40,41] as damping fluids to devise semi-active liquid dampers with alterable fluid viscosity. The ER fluids and the MR fluids are smart materials that can reversibly change from a free-flowing, linear viscous fluid to a semi-solid with controllable yield strength in milliseconds when exposed to an electric field, or a magnetic field respectively. The sharply alterable fluid viscosity results in adjustable and controllable damping forces for structural vibration control under a wide range of loading conditions. Fig 1.9 shows a schematic diagram of the controllable TLCD using MR-Fluid.

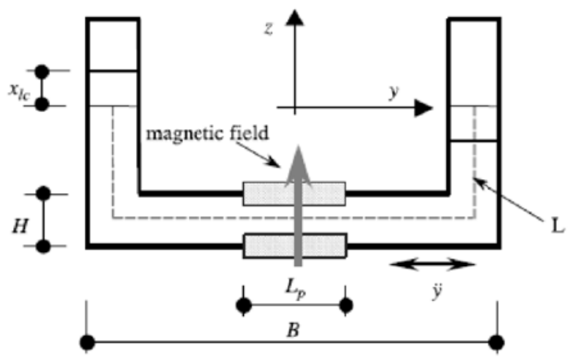


Fig. 9 (*schematic diagram of the MR-TLCD)*

1.4 PRACTICAL IMPLEMENTATION:

Till date, Tuned liquid dampers have been installed in number of the structures around the globe, from its first installation in 42 mtr high Nagasaki Airport Tower, Nagasaki, Japan in 1987 [16], which was a purely temporary installation, intended to verify the effectiveness of the TLD in reducing structural vibration. The actual measurements were exhortative one. It was found that, with installation of 25 vessels of TLD, the decrease in amplitude of vibration is 44 % (i.e from 0.79 mm without TLD to 0.44 mm) while reduction in RMS displacement was around 35%.

Similarly, in another experiment unveiled that, the maximum acceleration response of an uncontrolled Yokohama Marine Tower in Japan [16] under wind action was 0.27 m/s2, when the velocity of the wind was in the range of 15–21 m/s, the damping ratio was measured as 0.6%. But after using TLD as a vibration control device, the maximum acceleration response was reduced to 0.1 m/s2 or below, and the damping ratio was increased to 4.5%.

The TLD devices installed in the 77.6 m high structure of the Tokyo Airport Tower [16, 42] consist of 1400 tanks filled with water and floating polyethylene particles, which are added to enhance the energy dissipation. The containers, low cylinders of diameters 0.6 and 0.125 m, are stored in six layers on steel consoles. The total mass of the TLDs is approximately 3.5% of the first modal mass of the tower and its frequency is optimized to 0.74 Hz. The behavior of the TLD has been observed under various wind phenomena. In one of these observations, it was found that, with a maximum speed of 25 m/sec, the reduction in RMS acceleration was about 60% of its value without control.

Tuned Liquid Dampers has, successfully, been applied to some Bridge structures including Ikuchi Bridge, Japan, Bai Chay Bridge, Vietnam and Sakitama Bridge, Japan.

CHAPTER - 2 (LITERATURE REVIEW AND SCOPE OF THE WORK)

2. 1 BRIEF LITERATURE REVIEW:

Till date TLD has been studied by several researchers. Soong & Dargush [43] provide a comprehensive review of theoretical and experimental studies conducted on TLDs and structure-TLD system. Bauer [3] is the first to propose a damping device consisting of a liquid container filled with two immiscible liquids, in which the motion of the interface is able to dampen the structure effectively. Modi & Welt [4] were also among the first to suggest the use of a TLD in buildings to reduce overall response during strong wind or earthquakes. Due to geometrical similarity they referred these devices to Nutation dampers.

Fujii, et al.[5,16] have found by installing Wind-induced vibrations of two actual tall towers, at Nagasaki Airport Tower (height 42 m) and Yokohama Marine Tower (height 101 m), were reduced to about half upon installation of Tuned Sloshing Damper.

Sun, et al. [7] could successfully develop an analytical model for TLD, based on shallow water wave theory, which proved to be very effective if the wave is non breaking. They extended this model to account for effect of breaking waves by introducing two empirical coefficients identified experimentally. Wakahara, et al. [8] carried out theoretical and experimental studies to design an optimum TLD and verified the TLD with an actual application to a high-rise hotel the "Shin Yokohama Prince (SYP) Hotel" in Yokohama. The interaction model considered by them was based on the Boundary Element Method (BEM) for simulating liquid motion in a TLD container, and Multi-degrees of Freedom (MDOF) for the structure Method. The TLD installation on the building could reduce the wind-induced response to half the original value.

Sun, et al. [44] measured liquid motion in shallow TLDs, including rectangular, circular, and annular tanks subjected to harmonic base excitation. Using TMD analogy they calibrated the TLD parameters from experimental results. Chen et al. [45] conducted on pendulum-like testing model in order to simulate the long period motion of a high rise building. Experimentally, they conclude that the effeteness of the TLD system is significant compared with optimal TMD system or improved distributed TMD system. Also dynamic behavior of a TLD system is more similar to distributed TMD system than a TMD system.

Koh et al. [15] conducted numerical studies to investigate the effect combined use of liquid dampers which are tuned to different vibration frequencies of a multi-degree-of-freedom structure. The results show that it is beneficial to use dampers tuned to several vibration modes of the structure. Numerically they conclude that that, effectiveness of the dampers is dependent on the frequency content of the earthquake spectrum and the positions where the dampers are placed.

Tamura, et al. [16,42] have found that the damping ratio of 77.6m high Tokyo international Airport Tower have increased to 7.6% from 1.0% by using Tuned liquid Damper.

Modi & Seto [46] conducted numerical study on rectangular TLDs, accounting for nonlinear effects. They included the effects of wave dispersion as well as boundary-layers at the walls, floating particle interactions at the free surface, and wave-breaking. Modi & Munsi [47] conducted an experimental study to improve the TLD efficiency by introducing a two-dimensional obstacle. They conducted a parametric study to decide optimum size and location of the obstacle. Results suggested a significant increase in the energy dissipation, up to 60%, in the presence of the obstacle.

Reed et al. [48, 49] had conducted researches to investigate the effectiveness and robustness of a TLD over a large range of excitation amplitude. From his experiment he concluded that the response frequency of TLDs increases as excitation amplitude increases. Also this experiment revealed that, the maximum response occurs at a frequency higher than that estimated by the linearied water-wave theory. One consequence of this characteristic is that the TLD is robust in dissipating energy over a wide frequency range

Yu et al. [50] proposed a solid mass damper model which they referred as Non-linear-Stiffness-Damping (NSD) model, for the Tuned Liquid Damper with non-linear stiffness and damping. This model was an expansion of Tuned Mass Damper. They calibrated the non-linear characteristics of the NSD model from shaking table experiment tests.

Yamamoto & Kawahar [51] considered a fluid model using Navier-Stokes equation in the form of the arbitrary Lagrangian-Eulerian (ALE) Formulation. For the discretization of the incompressible Navier-Stokes equation, they used the improved-balancing-tensor-diffusivity method and the fractional-step method. For computational stability, smoothing on the free surface was carried out. Newmark's-β method was used for the time discritization. They used a numerical model of Yokohama Marine Tower to investigate effectiveness of the TLD model. They found that this model could effectively analyze the TLDs.

Chang & Gu [52] studied experimentally, the control effects of rectangular TLDs installed on a tall building that vibrates due to vortex excitation. They found that the rectangular TLD is quite effective in reducing the vortex-excited vibration of a building, especially when its frequency is tuned to within the optimal range. The top displacement RMS value reduces to only one-sixth of that of the original building model without a TLD when the generalized mass ratio equals to 2.3%. The optimal frequency of the TLD ranged between 0.9 and 1.0 of that of the building model which was consistent with the analytical derivation.

Kaneko & Ishikawa [53] conducted analytical study on TLD with submerged nets. They employed a liquid model based on nonlinear shallow water wave theory. Effect of the hydraulic resistance produced by the nets was examined. They verified the results of dissipation energy theoretically obtained by experiment. They found that the optimal damping factor, as in the case for TMDs, can be produced by nets, and the TLDs with submerged nets are more effective in reducing structural vibration than TLDs without.

Kaneko S. & Mizota Y. [54] expanded previously developed rectangular Deep water TLD model [53] to Cylindrical Deep water TLD model with a submerged net installed in the middle of the cylindrical liquid container. In the analysis, employing finite amplitude wave theory and Galerkin method in the case of cylindrical tank, they obtained hydrodynamic forces and the free surface elevations. Then, combining the hydrodynamic forces with the equation of motion of the structure, damped transient responses were calculated. The calculated results thus obtained were compared with the experimental results, by which the validity of the modeling methodology was confirmed.

Banarji et al. [55] used the formulation suggested by Sun et al. [7] in order to study the effectiveness of a rectangular TLD in reducing the earthquake response of structures for various values of natural time periods and structural damping ratios. Furthermore, an attempt is made to define appropriate design parameters of the TLD that is effective in controlling the earthquake response of a structure. These parameters include the ratio of the linear sloshing and structure natural frequencies, henceforth called the tuning ratio, the ratio of the masses of water and structure, henceforth called the mass ratio, and the water depth to the TLD tank-length ratio, henceforth called the depth ratio.

Gardarsson et al. [21] extended the idea of dramatically dissipation of tsunami wave energy, by the shores of an ocean coastline, to a TLD, by adding a slopped bottom tank. They experimentally calculated the sloshing characteristics associated with a slopped-bottom TLD and compared with a box-shaped TLD. The 300 Sloped bottom TLD considered for the study found to behaves like a softening spring, unlike to a box shaped TLD which behaves like a hardening spring. They found the sloped-bottom TLD is especially effective when it is tuned slightly higher than the structure's fundamental response frequency. However, they found some problem associated with slopped bottom TLD, as there will be a greater magnitude of the moment exerted at the TLD base. Olson & Reed [56] analytically studied a slopped bottom TLD proposed by Gardarsson et al. [21] using the non-linear stiffness and damping model developed by Yu et al [50]. The results clearly illustrated a system that is described by a softening spring.

Pal et al. [57] investigated the slosh dynamics of liquid-filled containers experimentally using a three-dimensional finite element analysis. The effects of sloshing were computed in the time domain using Newmark's time integration scheme. A simple experimental setup was designed to conduct experiments for measuring some of the basic parameters of sloshing. A sensor device was especially developed to record the free-surface wave heights.

Modi et al. [23] investigated on enhancing the energy dissipation efficiency of a rectangular liquid damper through introduction of two dimensional wedge shaped obstacles. From his experiment he concluded that wedging increases damping factor and damping factor further increases for a roughened wedge.

Li et al. [58] proposed a numerical model for the implementation of shallow rectangular TLD where the dynamic properties of shallow liquid in rectangular containers subjected to forced horizontal oscillations are analized directly from the continuity and momentum equations of fluid. Following some practical assumptions, they established the nonlinear partial differential equations describing the wave movement of shallow liquid in rectangular containers and proposed a numerical procedure for the solutions of these equations based on the finite element method.

Ikeda [59] investigated the nonlinear vibrations of a system, in which a rigid rectangular tank partially filled with liquid is attached to an elastic structure subjected to a vertical sinusoidal excitation. First, by taking into account the nonlinearity of fluid force, modal equations involving sloshing modes up to the third mode were derived when the natural frequency of the structure was equal to twice the frequency of sloshing of the liquid surface. Second, resonance curves for this system had been determined from the modal equations by using the harmonic balance method. Finally, the influences of the depth of the liquid and the detuning parameter on the resonance curves had been mainly investigated by showing the resonance curves. It was found that the shapes of the resonance curves markedly change depending on the liquid's depth, and that periodically and chaotically amplitude-modulated motions occur at certain intervals of the excitation frequency. Furthermore, it was also found that coupled vibrations can occur at two ranges of the excitation frequency when the deviation of the tuning condition is comparatively large. Finally he validated the numerical results with experimental results.

Casciati et al. [24] proposed a frustum-conical shaped TLD to an alternative to the traditional cylindrical tank. This allows calibrating the natural frequency through varying liquid depth, making it suitable for semi-active implementation and attains the same level of performance with a fewer mass. They presented a linear model which is only suitable for small excitations as for larger amplitudes, strong nonlinearities occur. They validated the linear model only for the case of harmonic excitations.

Biswal et al. [60] studied a two-dimensional finite element analysis for the dynamic analysis of liquid filled rectangular tank with baffles using the velocity potential formulation and the linear water wave theory. The slosh frequencies of liquid in a rectangular tank without and with baffles (thin rectangular plates) were evaluated. The tank-baffle system was considered to be rigid. The slosh response of liquid was studied under steady state sinusoidal base excitation. The slosh frequencies of liquid were computed for different dimensions and positions of baffle(s).

Biswal et al. [61] presented a free vibration analysis of liquid filled rigid cylindrical tank with annular baffles and compared the natural frequencies of liquid with that of tank without baffles. The slosh frequency parameters of liquid were computed for various locations of baffle in the tank. They observed that the baffle had appreciable effect on the slosh frequency parameters of liquid when placed very close to liquid-free surface for all R/R ratios. Further, the flexibility of baffle had an effect on the liquid slosh frequency parameters up to certain thickness of baffle.

Ikeda & Ibrahim [62] numerically analyzed an elastic structure carrying a cylindrical tank partially filled with liquid where the structure is vertically subjected to a narrow-band random excitation. They derived the modal equations taking into account the liquid nonlinear inertia forces. Nonlinear coupling between liquid modes and structure modes results in 2:1 internal resonance, i.e., when the natural frequencies of the structure and the first anti-symmetric sloshing mode were commensurable. They solved the modal equations numerically using Monte Carlo simulation, and estimated the system response statistics.

Tait et al. [19] discussed the numerical flow model of TLD behavior including the free surface motion the resulting base shear forces and the energy dissipated by TLD with slat screens. Both linear and nonlinear analytical models for TLD are examined and compared with experimental data. It was found that the linear model is capable of providing a first estimate of the energy dissipating characteristics of a TLD. However, the linear model could not provide realistic estimates of the free surface response for amplitudes experimentally investigated. The nonlinear model could accurately describe the free surface motion, the resulting base shear forces and the energy dissipated over a range of excitation amplitudes. The nonlinear model was capable of modeling a TLD equipped with multiple screens at various screen locations inside the tank. The nonlinear model was also verified over a range of practical fluid depth to tank length ratio values. They outlined a procedure to position and size the slat screens according to the linear model result.

Frandsen [63] adopted a fully nonlinear 2-D tank which is moved both horizontally and vertically by using σ-coordinate transformation for fluid model. He analyzed the model for various liquid heights corresponding to Deep water TLD and Shallow Water TLD. However, the above model is subjected to a limitation that, because of the use of potential flow assumption, both viscous sloshing and rotational motion of the liquid can't be captured by the models introduced above.

Kim et al. [12] conducted shaking table experiments to investigate the characteristics of water sloshing motion in TLD (rectangular and circular) and TLCD. They found the parameters such as wave height, base shear force, and energy dissipation etc. from the experiment. It was found that the TLCD was more effective in controlling vibration than TLD.

Tait et al. [64] studied the ability of TLD to operate in two directions. They conducted experimental test on bidirectional (2D) structure-TLD model and estimated the free-surface motion, the resulting base shear forces, and the work done by bidirectional tuned liquid dampers (2D TLD) attached to simple structures and response displacements and accelerations of 2D structure-TLD systems. The response of a 2D structure-TLD system excited bi-directionally was found to correspond to the linear superposition of the responses of two 1D structure-TLD systems. Findings from this study indicate that by choosing the appropriate aspect ratio for the TLD it can be used to reduce structural responses in two modes of vibration simultaneously with no penalty on its performance.

Jin et al. [65] studied the effectiveness of cylinder TLD in controlling earthquake response of jacket platform. They applied TLDs to a CB32A oil tank to prove the feasibility. They found that the ratio of the fundamental sloshing frequency of liquid to the natural frequency of platform is the key factor to control earthquake response. The larger ratio of water-mass to platform-mass is also useful to reduce vibration as well.

Lee et al. [66] proposed a real-time hybrid shaking table testing method (RHSTTM) to study the performance of a Tuned Liquid Damper (TLD) controlling seismically excited building structure. The RHSTTM model consists of an analytical building model and physical liquid model placed on a shaking table. The structural responses of the system, to a given earthquake load, were calculated numerically in real time from the analytical building model, and its state space realization incorporated in the integrated controller of the shaking table, to calculate shear force generated by the TLD. They validated the structural responses obtained by the RHSTTM and the conventional shaking table test of a single storey steel frame with TLD, and found that the performance of the TLD can be accurately evaluated using the RHSTTM without the physical structural model.

M.J. Tait [20] could successfully develop an equivalent linear mechanical model that accounts for the energy dissipated by the damping screens. He developed expressions for equivalent damping ratio expressions for both sinusoidal and random excitation. He further extended this to outline a rapid preliminary design procedure initial TLD sizing and initial damping screen design for a TLD equipped with damping screens.

Attari & Rofooei [67] studied nonlinear interaction between a SDOF structural system, assumed to be an idealization of MDOF structural system, carrying a circular cylindrical liquid tank and the sloshing modes of the liquid is investigated. The system was considered nonlinear due to the convective term of liquid acceleration and the nonlinear surface boundary conditions, both caused by the inertial nonlinearity. Response of this model under horizontal harmonic and earthquake excitations was studied using 1 and 3 sloshing modes in the neighborhood of 1:2 and 1:1 internal resonances. They also studied the energy transfer from the structural mode to the first unsymmetrical sloshing mode of liquid.

Shang & Zhao [13] numerically studied effect of two angle-adjustable baffles, in a rectangular TLD. The fundamental natural periods of the damper can be changed in a wide range by adjusting the baffle angles, thus making it more effective in controlling the vibration of structures in a wide frequency range. The amount of eddies could be influenced by the angles of the baffles. They found that the TLD tank is much more effective in reducing the earthquake responses of structure than the same size general rectangular TLD tank.

Marivani, et al. [68] developed an integrated fluid-structure numerical model to simulate the response of an SDOF system outfitted with a TLD. The fluid flow model was a two-dimensional non linear model. He successfully validated his model using three fluid flow problems. Further the fluid-structure model was tested; whose results show that the model was capable of capturing the right expected damping effect of the TLD on the structure response.

2. 2 AIM AND SCOPE OF THIS WORK:

Thus far, there have been numerous studies on implementation of TLD for structural vibration control. However, most of the studies have assumed the structural model either as a SDOF model or 1D-MDOF model. To the best of my knowledge, no has considered the structural model a 2D-MDOF system. Similarly, very few works have been done on application of TLD to control vibration of structures during earthquakes. Therefore, these two unexplored areas has become my aim of study.

The scope of the present study included the development of a fluid-structure interaction model, to simulate the response of a 2D-MDOF fitted with a Tuned Liquid Damper (TLD). A 2D-TLD model using rectangular tank has been considered for the present study. Two sinusoidal ground acceleration case and five random ground acceleration cases corresponding to past earthquake time histories are applied to the Fluid-structure interaction model.

The problem has been studied in three parts; firstly, the responses of the structure to the above mentioned loading conditions are studied. Secondly, the response of the fluid model to the above mentioned loading conditions has been studied; finally, the coupled Fluid-Structure interaction model has been studied. The effectiveness of the TLD in controlling the vibration is measures in terms of amplitude of controlled displacement of top storey of the structure using TLD and amplitude of uncontrolled displacement of top storey of the structure without using TLD.

CHAPTER - 3 (MATHEMATICAL FORMULATIONS)

3. 1 ASSUMPTIONS:

- The liquid is considered homogeneous, irrotational and incompressible.
- The walls of the damper are treated rigid.
- The liquid surface is remains smooth during sloshing (No breaking wave produced)
- It should be emphasized that the structural response acts as an excitation for the damper thus affecting the sloshing motion of the liquid and its dissipation. On the other hand, the nutation damping affects the structural response. The analysis accounts for this conjugate character.

3. 2 GOVERNING EQUATION FOR THE LIQUID

The governing differential equation in terms of pressure variable is

$$\nabla^2 P = 0 \qquad \text{On V} \tag{3.1}$$

Where, V is the volume of liquid domain

$P = P(x, y, z, t)$ is the liquid dynamic pressure,

And, $\nabla^2 = \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2} + \frac{\partial^2}{\partial z^2}$ in Cartesian coordinate system

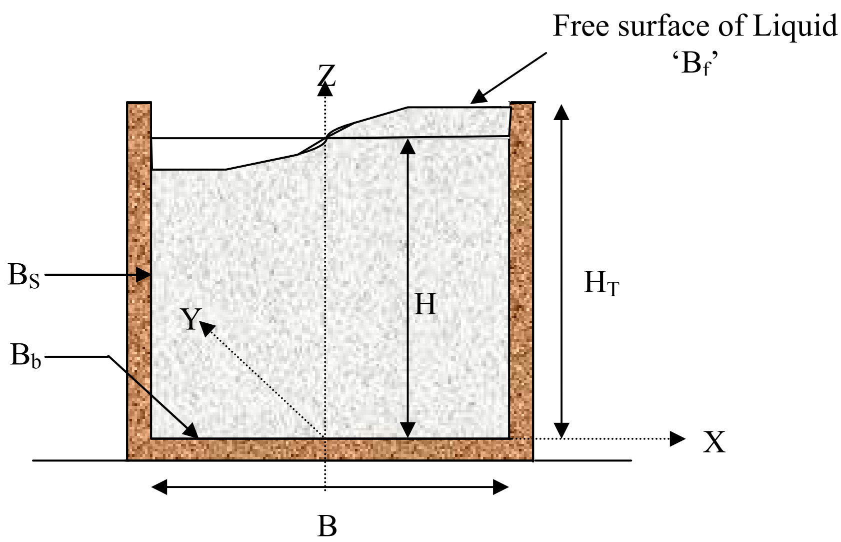


Fig 3.1 (Dimensions of a Rectangular Tank)

3.2.1 Boundary Conditions:

The liquid domain is bounded by the structure boundary and free liquid surface boundary.

a) At liquid free surface:

Two boundary conditions are imposed at the liquid free surface. One is the kinematic boundary condition, which states that a liquid particle on the free surface at all the time will always be on the free surface and the second one is dynamic boundary condition, which specifies that the pressure on the free surface is zero. The latter is implemented through the Bernoulli's equation for unsteady and irrotational motion. By considering the small amplitude waves, the velocity term in the Bernoulli's equation may be neglected. Both the kinematic and dynamic boundary conditions may be combined to get the single linearlized free surface equation as stated below:

$$\frac{\partial^2 P}{\partial t^2} + g\frac{\partial P}{\partial n} = 0 \qquad \text{on } B_f \qquad (3.2)$$

Here B_f = Free surface area.

However, if free surface wave of the liquid is ignored, then P = 0.

b) At liquid surface Interface:

$$\frac{\partial P}{\partial n} = -\rho_f \ddot{d}_n \qquad \text{on } B_s \qquad (3.3)$$

Where, $\ddot{d}_n$ is the acceleration of the structure, and

n is outwardly drawn normal to the surface of the structure.

Bs the surface area of the liquid in contact with the tank wall and block.

c) At Bottom of the Tank:

$$\frac{\partial P}{\partial n} = 0 \qquad \text{on } B_b \qquad (3.4)$$

Where, Bb is the tank bottom

3.2.2 Finite Element Formulation

For the case of a rectangular container, the liquid dynamic pressure ‘P’ is approximated as

$$P(x,y,z,t)=\sum_{j=1}^{N} N_j(x,y,z)\overline{P}_j(t) \tag{3.5}$$

In which N_j are the shape functions and $\overline{P}_j(t)$ are the time dependent nodal pressures.

Applying divergence theorem to the residual form of governing differential equation for the liquid and minimizing the energy function, one obtains:

$$\int_V \left(\frac{\partial N_i}{\partial x}\sum_1^N \frac{\partial N_j}{\partial x}\overline{P}_j+\frac{\partial N_i}{\partial y}\sum_1^N \frac{\partial N_j}{\partial y}\overline{P}_j+\frac{\partial N_i}{\partial z}\sum_1^N \frac{\partial N_j}{\partial z}\overline{P}_j\right)dV=\int_B N_i\frac{\partial P}{\partial n}ds \tag{3.6}$$

In which $B = B_f + B_s + B_b$

Where, B_f, B_s and B_b are defined in Figure 3.1

Substituting equations (3.2), (3.3) and (3.4) in equation (3.6) we will get,

$$\int_V \left(\frac{\partial N_i}{\partial x}\sum_1^N \frac{\partial N_j}{\partial x}\overline{P}_j+\frac{\partial N_i}{\partial y}\sum_1^N \frac{\partial N_j}{\partial y}\overline{P}_j+\frac{\partial N_i}{\partial z}\sum_1^N \frac{\partial N_j}{\partial z}\overline{P}_j\right)dV=-\int_{B_S}\rho_f N_i\ddot{d}_n ds-\frac{1}{g}\int_{B_f}N_i\sum_1^N N_j\ddot{\overline{P}}_j ds \tag{3.7}$$

We may reduce equation 3.7 to

$$[M_f]\{\ddot{P}\}+[K_f]\{P\}=\{F_p\} \tag{3.8}$$

Here elements $[M_f]$, $[K_f]$ and $\{F_p\}$ are given by

$$M_{ij}=\frac{1}{g}\sum\int_{B_f}N_iN_j\ ds \tag{3.9}$$

$$K_{ij}=\sum\int_V\left[\frac{\partial N_i}{\partial x}\frac{\partial N_j}{\partial x}+\frac{\partial N_i}{\partial y}\frac{\partial N_j}{\partial y}+\frac{\partial N_i}{\partial z}\frac{\partial N_j}{\partial z}\right]dV \tag{3.10}$$

$$F_i=-\sum\int_{B_s}\rho_f N_i\ddot{d}_n\ ds \tag{3.11}$$

3.2.3 Two-Dimensional Approach

a) Liquid Stiffness Matrix

A two-dimensional finite element analysis is adopted for the sloshing analysis of liquid filled rectangular tank with baffles. The liquid domain is discretized by 4-noded quadrilateral liquid elements.

The stiffness matrix of the liquid domain is computed by summation of element stiffness matrices, i.e.

$$[K_f] = \sum_{s=1}^{NK} K_s \tag{3.12}$$

Where NK is the number of liquid elements and element stiffness matrix K_s is given by

$$K_s = \int_\Omega \left[\frac{\partial N_i}{\partial x}\frac{\partial N_j}{\partial x} + \frac{\partial N_i}{\partial z}\frac{\partial N_j}{\partial z} \right] d\Omega \tag{3.13}$$

In a two-dimensional field problem and in case of a Cartesian coordinate system, the dynamic pressure in a liquid element may be as approximated by the following equation.

$$P_s(x,z,t) = \sum_{j=1}^{4} N_j(x,z)\ \ \overline{P}_{js}(t) \tag{3.14}$$

By taking the derivative of equation (3.14) separately with respect to x and z, we obtain the liquid pressure gradient within the s^{th} liquid element as expressed below.

$$\begin{Bmatrix} \dfrac{\partial \overline{P}}{\partial x} \\ \dfrac{\partial \overline{P}}{\partial z} \end{Bmatrix}_s = \begin{Bmatrix} \dfrac{\partial N_j}{\partial x} \\ \dfrac{\partial N_j}{\partial z} \end{Bmatrix} \{\overline{P}_j\}_s \tag{3.15}$$

The derivative of shape function with respect to the natural coordinates (ξ, η) is obtained through the Jacobian transformation using the chain rule of differentiation.

$$\begin{Bmatrix} \dfrac{\partial N_j}{\partial \xi} \\ \dfrac{\partial N_j}{\partial \eta} \end{Bmatrix} = \begin{Bmatrix} \dfrac{\partial x}{\partial \xi} & \dfrac{\partial z}{\partial \xi} \\ \dfrac{\partial x}{\partial \eta} & \dfrac{\partial z}{\partial \eta} \end{Bmatrix} \begin{Bmatrix} \dfrac{\partial N_j}{\partial x} \\ \dfrac{\partial N_j}{\partial z} \end{Bmatrix}$$

(3.16)

or

$$\begin{Bmatrix} \dfrac{\partial N_j}{\partial \xi} \\ \dfrac{\partial N_j}{\partial \eta} \end{Bmatrix} = [J_s] \begin{Bmatrix} \dfrac{\partial N_j}{\partial x} \\ \dfrac{\partial N_j}{\partial z} \end{Bmatrix} \tag{3.17}$$

where

$$[J_s] = \begin{Bmatrix} \dfrac{\partial x}{\partial \xi} & \dfrac{\partial z}{\partial \xi} \\ \dfrac{\partial x}{\partial \eta} & \dfrac{\partial z}{\partial \eta} \end{Bmatrix} \text{, the Jacobian matrix} \tag{3.18}$$

$$\begin{Bmatrix} \dfrac{\partial N_j}{\partial x} \\ \dfrac{\partial N_j}{\partial z} \end{Bmatrix} = [J_s]^{-1} \begin{Bmatrix} \dfrac{\partial N_j}{\partial \xi} \\ \dfrac{\partial N_j}{\partial \eta} \end{Bmatrix} = [J_s]^{-1}\, T_s(\xi,\eta) \tag{3.19}$$

In which $[J_s]^{-1}$ is the inverse of the Jacobian matrix and

$T_s(\xi,\eta)$ contains the derivative of shape functions.

Let

$$\begin{Bmatrix} \dfrac{\partial N_j}{\partial x} \\ \dfrac{\partial N_j}{\partial z} \end{Bmatrix} = [B] \tag{3.20}$$

where $[B] = [J_s]^{-1}\, T_s(\xi,\eta)$.

The liquid element stiffness matrix in equation (3.13) may be written as

$$K_s = \int_{\Omega} \begin{bmatrix} \dfrac{\partial N_i}{\partial x} & \dfrac{\partial N_i}{\partial z} \end{bmatrix} \begin{bmatrix} \dfrac{\partial N_j}{\partial x} \\ \dfrac{\partial N_j}{\partial z} \end{bmatrix} d\Omega$$

$$= \int_{\Omega} [B]^T [B] \; dx \; dz \; = \int_{-1}^{+1} \int_{-1}^{+1} [B]^T [B] \, |J_s| \; d\xi \; d\eta \qquad (3.21)$$

b) Liquid Free Surface Mass Matrix

The liquid free surface mass matrix for the system is formed by the assembly of line elements in the liquid surface, i.e.

$$[M_f] = \sum_{t=1}^{NF} M_{ef} \qquad (3.22)$$

Where NF is the number of liquid elements and M_{ef} is given by

$$M_{ef} = \frac{1}{g} \int_{B_f} N_i N_j \; ds \qquad (3.23)$$

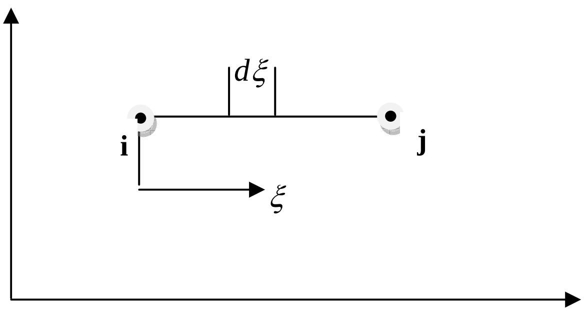


Figure 3.2 Liquid free surface elements in rectangular container

The liquid free surface element is shown in Figure 3.2, where ξ the local coordinate for the free surface boundary element and L is the length of the element.

The shape functions for the line element at the free surface may be defined as

$$N_j = \begin{bmatrix} 1 - \dfrac{\xi}{L} & \dfrac{\xi}{L} \end{bmatrix} \qquad (3.24)$$

The liquid free surface matrix in equation (3.23) may be written as

$$M_{ef} = \frac{1}{g} \int_0^L \begin{bmatrix} 1-\dfrac{\xi}{L} \\ \dfrac{\xi}{L} \end{bmatrix} \begin{bmatrix} 1-\dfrac{\xi}{L} & \dfrac{\xi}{L} \end{bmatrix} d\xi \qquad (3.25)$$

c) Liquid Load Vector

The load vector for the system is given by

$$[F_f] = \sum_{t=1}^{NL} F_{ef} \qquad (3.26)$$

Where NL is the number of elements at the liquid-structure interface and element load vector F_{ef} is given by

$$F_{ef} = -\int_{B_s} \rho_f N_i \ddot{d}_n \; ds \qquad (3.27)$$

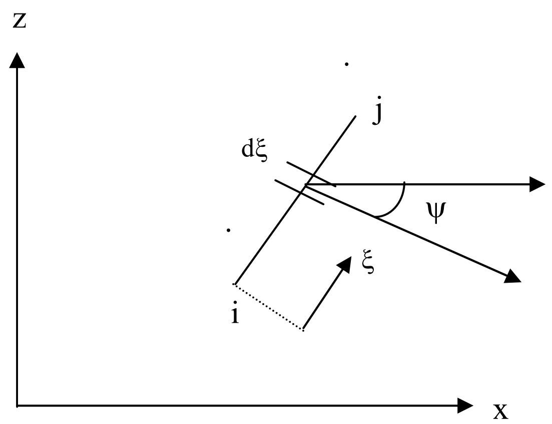


Figure 3.3 (Liquid-solid interface element in rectangular container)

The liquid-solid interface element is illustrated in Figure 3.3. The shape functions for the liquid-solid interface element is expressed as

$$N_j = \begin{bmatrix} 1-\dfrac{\xi}{L} & \dfrac{\xi}{L} \end{bmatrix} \qquad (3.28)$$

If $\ddot{d}_x$ is the horizontal ground acceleration in the x-direction then the normal acceleration at the liquid solid interface may be expressed as

$$\ddot{d}_n = \ddot{d}_x \cos\psi \qquad (3.29)$$

where

$$\psi = \tan^{-1}(x_i - x_j)/(z_i - z_j).$$

The load vector in equation (3.27) may be written as

$$F_t = -\int_0^L \rho_f \begin{Bmatrix} 1-\dfrac{\xi}{L} \\ \dfrac{\xi}{L} \end{Bmatrix} \ddot{d}_x \cos\psi \, d\xi \qquad (3.30)$$

3. 3 FORCED VIBRATION ANALYSIS OF LIQUID:

When some external force is applied to the liquid, then the finite element formulation for the liquid in the rigid tank may be expressed as

$$[M_f]\{\ddot{P}\} + [K_f]\{P\} = \{F_{el}\} \qquad (3.31)$$

Where, $\{F_{el}\}$ is the external nodal load vector and given by,

$$[F_{el}] = \sum_{t=1}^{NL} F_{el}$$

Where, $$[F_{el}] = -\rho_f \int_{B_s} N_i^{\,T} \ddot{d}_n ds$$

$$= -\rho_f \ddot{d}_n \int_{-1}^{+1}\int_{-1}^{+1} N_i^{\,T} |J_s| d\zeta d\eta \qquad (3.32)$$

3.3.1 Solution of Forced vibration problem using Newmark Beta Method:

The forced vibration problem can be solved by Newmark Beta Method, also known as the constant average acceleration method.

For the solution of the displacement, velocity and acceleration at time t+Δt are also considered,

$$[M_f]^{t+\Delta t}\{\ddot{P}\} + [K_f]^{t+\Delta t}\{P\} = \{F_{el}\}^{t+\Delta t}$$

The algorithm of the scheme is highlighted below:

A. Initial calculations:

1. Formulation of Global Stiffness matrix K and Mass matrix M
2. Initialization of P and $\ddot{P}$
3. Selection of time step Δt and parameters $\beta^{'}$ and $\alpha^{'}$

$$\alpha' \geq 0.5 \text{ and } \beta' \geq 0.25(0.5+\alpha')^2$$

$\alpha' = 0.5$ and $\beta' = 0.25$ are taken in the present analysis

4. Calculation of coefficients for the time integration

$$a_0 = \frac{1}{\beta'\Delta t^2}; a_1 = \frac{\alpha'}{\beta'\Delta t}; a_2 = \frac{1}{\beta'\Delta t}; a_3 = \frac{1}{2\beta'} - 1;$$
$$a_4 = \frac{\alpha'}{\beta'} - 1; a_5 = \frac{\Delta t}{2}\left(\frac{\alpha'}{\beta'} - 2\right); a_6 = \Delta t(1-\alpha'); a_7 = \alpha'\Delta t$$

5. Computation of effective stiffness matrix $\hat{K}$

$$\hat{K} = K + a_0 M$$

B. For each time step:

1. Calculation of effective load vector

$$\hat{F}_{t+\Delta t} = F_{t+\Delta t} + M(a_0 P_t + a_2 \dot{P}_t + a_3 \ddot{P}_t)$$

2. Solution for pressure at time t+Δt

$$\hat{K}P_{t+\Delta t} = \hat{F}_{t+\Delta t}$$

3. Calculation of time derivatives of pressure (P) at time t+Δt

$$\ddot{P}_{t+\Delta t} = a_0(P_{t+\Delta t} - P_t) - a_2\dot{P}_t - a_3\ddot{P}_t$$
and
$$\dot{P}_{t+\Delta t} = \dot{P}_t + a_6\ddot{P}_t + a_7\ddot{P}_{t+\Delta t}$$

3. 4 THE FUNDAMENTAL SLOSHING FREQUENCY OF A TLD

The fundamental sloshing frequency of a TLD, f_{TLD} can be estimated using linear water sloshing frequency, f_w Ca, given by

$$f_w = \frac{1}{2\pi}\sqrt{\frac{\pi g}{h}\tanh\left(\frac{\pi h}{L}\right)} \qquad (3.33)$$

Where, g = acceleration due to gravity,

h = still water depth,

L = length of tank in the direction of sloshing motion.

The sloshing frequency, f_{TLD} is amplitude dependant; However, for small sloshing amplitudes we can assume $f_{TLD} = f_w$

3. 5 THE FLUID–STRUCTURE INTERACTION MODEL

The 2D frame model is discretized into a number of elements, as shown in the fig 3.4. We can consider infinite numbers of nodes in each element. Three degrees of freedom i.e, two translations and one rotation is associated to each node.

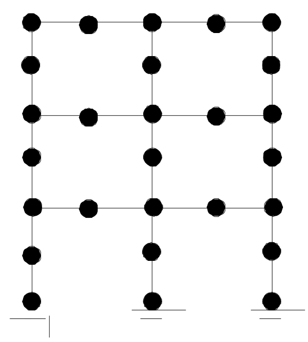


Figure 3.4 (Finite element discretization of Plane Frame)

For frame structure the equation of motion of this fluid–structure system can be written in the following form.

$$[M]\{\ddot{X}\} + [C]\{\dot{X}\} + [K]\{X\} = -[M]\{\ddot{X}_g\} + \{F_{TLD}\} \qquad (3.34)$$

Where,

[M] = The global mass matrix of the 2D frame structure

[C] = The global damping matrix of the frame structure (Assumed to be a zero matrix, as damping is neglected in the structure)

[K] = The global stiffness matrix of the 2D frame structure

{X} = The global nodal displacement vector

$\{\ddot{X}_g\}$= Ground Acceleration

$\{F_{TLD}\}$ = Resisting force to the structure at corresponding nodes due to TLD

a) Element Matrices:

The element stiffness matrix for a frame structure is given by:

$$[k]=\begin{bmatrix} \frac{EA}{L} & 0 & 0 & -\frac{EA}{L} & 0 & 0 \\ 0 & \frac{12EI}{L^3} & \frac{6EI}{L^2} & 0 & -\frac{12EI}{L^3} & \frac{6EI}{L^2} \\ 0 & \frac{6EI}{L^2} & \frac{4EI}{L} & 0 & -\frac{6EI}{L^2} & \frac{2EI}{L} \\ -\frac{EA}{L} & 0 & 0 & \frac{EA}{L} & 0 & 0 \\ 0 & -\frac{12EI}{L^3} & -\frac{6EI}{L^2} & 0 & \frac{12EI}{L^3} & -\frac{6EI}{L^2} \\ 0 & \frac{6EI}{L^2} & \frac{2EI}{L} & 0 & -\frac{6EI}{L^2} & \frac{4EI}{L} \end{bmatrix} \quad (3.35)$$

Where E = Young's Modulus of the frame element.

A = Cross sectional area of the element.

L = Length of the element.

The element mass matrix for a frame structure is given by:

$$[m]=\begin{bmatrix} 2a & 0 & 0 & a & 0 & 0 \\ 0 & 156b & 22l^2b & 0 & 54b & -13lb \\ 0 & 22l^2b & 4l^2b & 0 & 13lb & -3l^2b \\ a & 0 & 0 & 2a & 0 & 0 \\ 0 & 54b & 13lb & 0 & 156b & -22lb \\ 0 & -13lb & -3l^2b & 0 & -22lb & 4l^2b \end{bmatrix} \quad (3.36)$$

Where , $a=\frac{\rho AL}{6}$ and $b=\frac{\rho AL}{420}$

ρ = Density of the material.

b) Element Matrices Global Coordinate System:

The matrices formulated in the previous section are for a particular frame element in a specific orientation. A full frame structure usually comprises numerous frame elements of different orientations joined together. As such, their local coordinate system would vary from one orientation to another. To assemble the element matrices together, all the matrices must first be expressed in a common coordinate system, which is the global coordinate system.

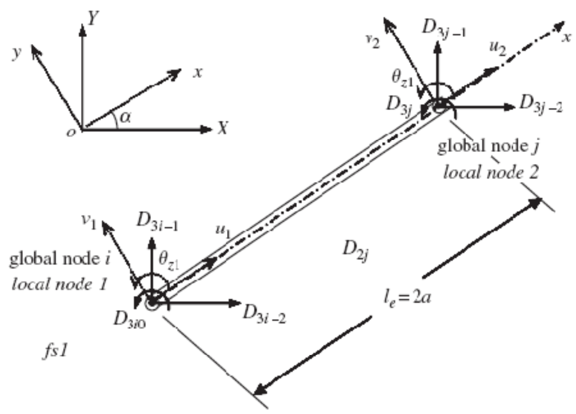


Fig. 3.5 (Coordinate transformation for 2D frame elements)

Assume that local nodes 1 and 2 correspond to the global nodes i and j, respectively. Let and T is the transformation matrix for the frame element given by

$$[T] = \begin{bmatrix} l_x & m_x & 0 & a & 0 & 0 \\ l_y & m_y & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & l_x & m_x & 0 \\ 0 & 0 & 0 & l_y & m_y & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix} \qquad (3.37)$$

Where:

$$l_x = \cos(x, X) = \cos\alpha = \frac{X_j - X_i}{l_e}$$

$$m_x = \cos(x, Y) = \sin\alpha = \frac{Y_j - Y_i}{l_e}$$

$$l_y = \cos(y, X) = \cos(90 + \alpha) = -\sin\alpha = -\frac{Y_j - Y_i}{l_e}$$

$$l_y = \cos(y, Y) = \cos\alpha = \frac{X_j - X_i}{l_e}$$

Here, α = the angle between x-axis and the X-X axis, as shown in the fig 3.5

$$l_e = \sqrt{(X_j - X_i)^2 - (Y_j - Y_i)^2}$$

Using the transformation matrix, T, the matrices for the frame element in the global coordinate system become

$$K_e = T^T kT$$
$$M_e = T^T mT$$

c) Application of Boundary Condition:

The boundary conditions are imposed on the structure by cancellation of the corresponding rows and columns in the stiffness as well as in mass matrix.

3. 6 FORCED VIBRATION ANALYSIS OF INTERACTION PROBLEM:

The equation 3.34 is a coupled one containing a number of independent variables. These coupled equations can be uncoupled to into a set of differential equations in which each equation contains only one dependant variable. For this purpose we can express the displacement {X} in terms of natural modes of system without damping; as

$$\{X\} = \Phi\ \{q\} \tag{3.38}$$

Where, Φ is the normalized modal matrix.

Substituting eqn (3.38) in eqn (3.34):

$$[M]\Phi\{\ddot{q}\} + [C]\Phi\{\dot{q}\} + [K]\Phi\{q\} = \{F_e\} - \{F_{TLD}\} \tag{3. 39}$$

Pre-multiplying by Φ^T gives

$$\Phi^T[M]\Phi\{\ddot{q}\} + \Phi^T[C]\Phi\{\dot{q}\} + \Phi^T[K]\Phi\{q\} = \Phi^T\{F_e\} - \Phi^T\{F_{TLD}\}$$

Or, $$\hat{M}\{\ddot{q}\} + \hat{C}\{\dot{q}\} + \hat{K}\{q\} = P(t) \tag{3.40}$$

Where,

$\hat{M}$ = Modal mass matrix

$\hat{C}$ = Modal damping matrix

$\hat{K}$ = Modal stiffness matrix

$P(t)$ = Modal force vector

q = displacement in normal coordinate

In equation (3.40) $\hat{M}$ & $\hat{K}$ are diagonal matrices and $\hat{C}$ is diagonal matrix for *classical damped* system.

Hence for a classically damped system the equation (3.40) can be uncoupled to into a set of differential equations:

$$\left.\begin{array}{l}\hat{M}(1,1).\ddot{q}(1)+\hat{C}(1,1).\dot{q}(1)+\hat{K}(1,1).q(1)=P(1)\\ \hat{M}(2,2).\ddot{q}(2)+\hat{C}(2,2).\dot{q}(2)+\hat{K}(2,1).q(2)=P(2)\\ ------------------------\\ \hat{M}(n,n).\ddot{q}(n)+\hat{C}(n,n).\dot{q}(n)+\hat{K}(n,n).q(n)=P(n)\end{array}\right\} \tag{3.41}$$

Solution of each set of these uncoupled equn can be found using Duhamel's integral method.

Displacements in absolute coordinate can be calculated from normal coordinates:

$$\{X\} = \Phi^T\{q\} \tag{3.42}$$

3.6.1 Numerical Evaluation of Duhamel's Integral-Damped System:

The Duhamel integral method has been used to solve the equation of motion of the structure. This integral method can be used to determine the response of the SDOF system under any type of external excitations (harmonic and random). The total displacement of a damped SDOF system subjected to an arbitrary external force can be expressed as:

$$X(t)=e^{-\xi\omega_n t}\left(x_0\cos\omega_D t+\frac{u_0+X_0\xi\omega_n}{\omega_D}\sin\omega_D t\right) \tag{3.43}$$

Setting $x_0 = 0$, and $u_0 = F_t(\tau)d\tau / M_s$ and substituting t for t- τ in Eq. (3.43) and integration of this equation over the entire loading interval results in:

$$X(t)=\frac{1}{M_s\omega_D}\int_0^t F_t(\tau)e^{-\xi\omega_n(t-\tau)}\sin\omega_D(t-\tau)d\tau \tag{3.44}$$

Where, F_t is the sum of the external excitation force and the TLD sloshing force. *X(t)* in Eq. (3.44) is the response of a damped system using Duhamel's integral. The displacement is calculated by numerical integration of Eq. (3.44). Using the trigonometric identity $\sin\omega(t-\tau)=\sin\omega t\cos\omega\tau-\cos\omega t\sin\omega\tau$ the displacement can be calculated from:

$$X(t)=\frac{e^{-\xi\omega_n t}}{M_s\omega_D}\{A_D(t)\sin\omega_D t-B_D(t)\cos\omega_D t\} \tag{3.45}$$

Where, A_D and B_D can be evaluated from:

$$A_D(t_i)=A_D(t_{i-1})+\int_{t_{i-1}}^{t_i}F_t(\tau)e^{\xi\omega_n\tau}\cos(\omega_D\tau)d\tau \tag{3.46}$$

$$B_D(t_i)=B_D(t_{i-1})+\int_{t_{i-1}}^{t_i}F_t(\tau)e^{\xi\omega_n\tau}\sin(\omega_D\tau)d\tau \tag{3.47}$$

Considering a linear piecewise loading function, the forcing function $F_t(\tau)$ can be approximated by:

$$F_t(\tau)=F_t(t_{i-1})+\frac{\Delta F_i}{\Delta t_i}(\tau-t_{i-1}), \qquad t_{i-1}\le\tau\le t_i \tag{3.48}$$

Where,

$$\Delta F_i=F_t(t_i)-F_t(t_{i-1}) \tag{3.49}$$

$$F_t(t_i)=F_e(t_i)+F_{TLD}(t_i) \tag{3.50}$$

$$\Delta t_i=i_i-t_{i-1} \tag{3.51}$$

In Eqn. (3.46), (3.47), $A_D(t_i)$ and $B_D(t_i)$ can be evaluated from:

$$A_D(t_i)=A_D(t_{i-1})+\left(F_t(t_{i-1})-t_{i-1}\frac{\Delta F_i}{\Delta t_i}\right)I_1+\frac{\Delta F_i}{\Delta t_i}I_1 \tag{3.52}$$

$$B_D(t_i)=B_D(t_{i-1})+\left(F_t(t_{i-1})-t_{i-1}\frac{\Delta F_i}{\Delta t_i}\right)I_2+\frac{\Delta F_i}{\Delta t_i}I_3 \tag{3.53}$$

Where the integrals I_1, I_2, I_3 and I_4, are evaluated as follows:

$$I_1=\int_{t_{i-1}}^{t_i}e^{\xi\omega_n\tau}\cos\omega_D\tau d\tau=\frac{e^{\xi\omega_n\tau}}{(\xi\omega_n)^2+\omega_D^2}(\xi\omega_n\cos\omega_D\tau+\sin\omega_D\tau)\Bigg|_{t_{i-1}}^{t_i} \tag{3.54}$$

$$I_2 = \int_{t_{i-1}}^{t_i} e^{\xi\omega_n\tau} \sin\omega_D\tau d\tau = \frac{e^{\xi\omega_n\tau}}{(\xi\omega_n)^2 + \omega_D^2}(\xi\omega_n \sin\omega_D\tau + \omega_D \cos\omega_D\tau)\Bigg|_{t_{i-1}}^{t_i} \tag{3.55}$$

$$I_3 = \int_{t_{i-1}}^{t_i} \tau.e^{\xi\omega_n\tau} \sin\omega_D\tau d\tau = \tau - \frac{\xi\omega_n}{(\xi\omega_n)^2 + \omega_D^2} I_2 + \frac{\omega_D}{(\xi\omega_n)^2 + \omega_D^2} I_1\Bigg|_{t_{i-1}}^{t_i} \tag{3.56}$$

$$I_4 = \int_{t_{i-1}}^{t_i} \tau.e^{\xi\omega_n\tau} \cos\omega_D\tau d\tau = \tau - \frac{\xi\omega_n}{(\xi\omega_n)^2 + \omega_D^2} I_1 + \frac{\omega_D}{(\xi\omega_n)^2 + \omega_D^2} I_2\Bigg|_{t_{i-1}}^{t_i} \tag{3.57}$$

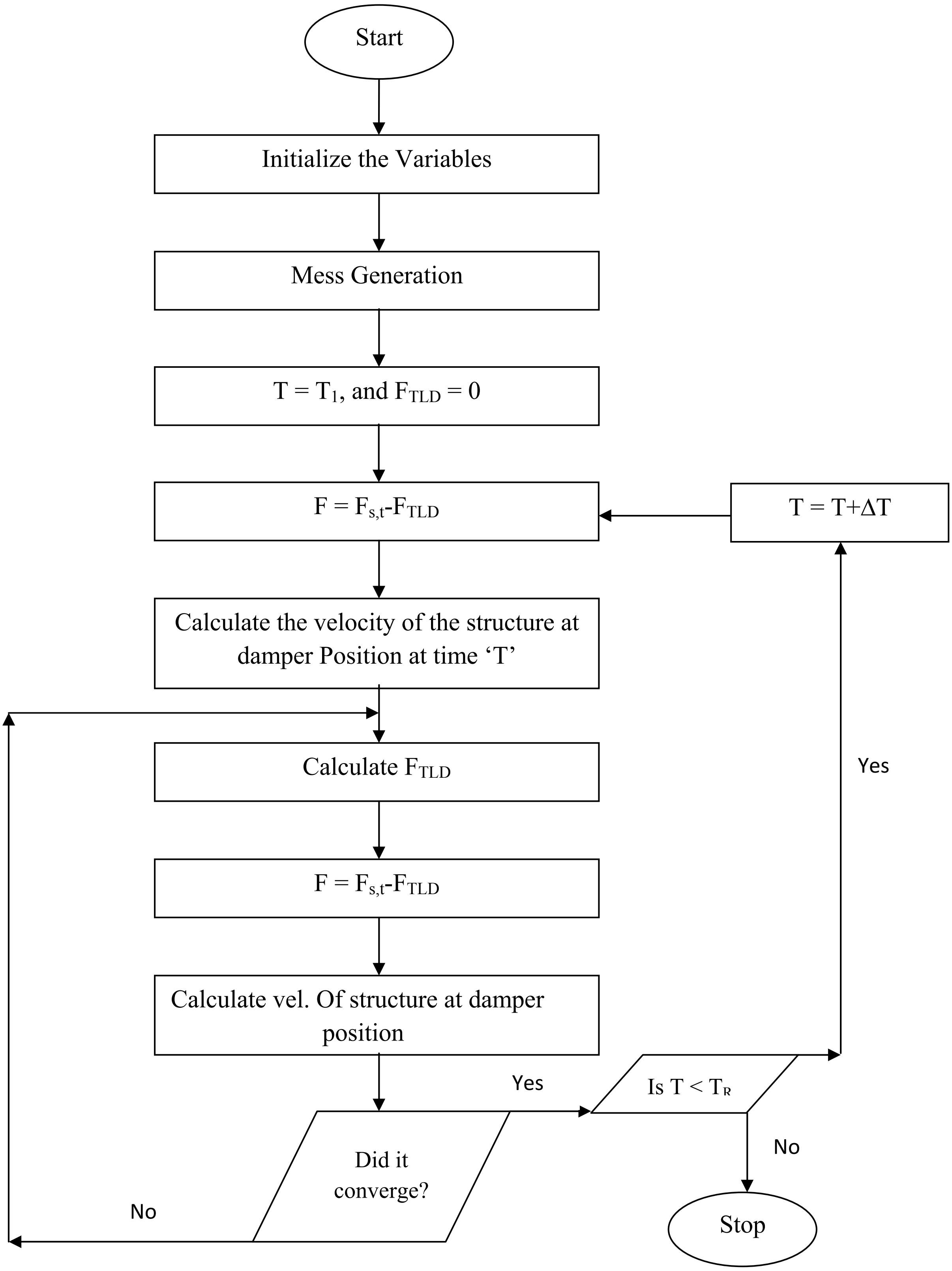
The substitution of Eqn. (3.49) and (3.50) into Eq. (3.45) gives the displacement at time t_i as:

$$X(t_i) = \frac{e^{-\xi\omega_n t_i}}{M_s\omega_D}\{A_D(t_i)\sin\omega_D t_i - B_D(t_i)\cos\omega_D t_i\} \tag{3.58}$$

The derivative of the above equation with respect to time t_i gives the velocity at time t_i as:

$$V(t_i) = \frac{e^{-\xi\omega_n t_i}}{M_s\omega_D}\{(B_D\omega_D - A_D e\omega_n)\sin\omega_D t_i - (A_D\omega_D + B_D e\omega_n)\cos\omega_D t_i\} \tag{3.59}$$

3. 7 FLOW CHART FOR FLUID –STRUCTURE INTERACTION PROBLEM:



CHAPTER - 4 (RESULTS AND DISCUSSION)

4.1 PROBLEM STATEMENT:

A 10 storey framed structure has been considered for the analysis.

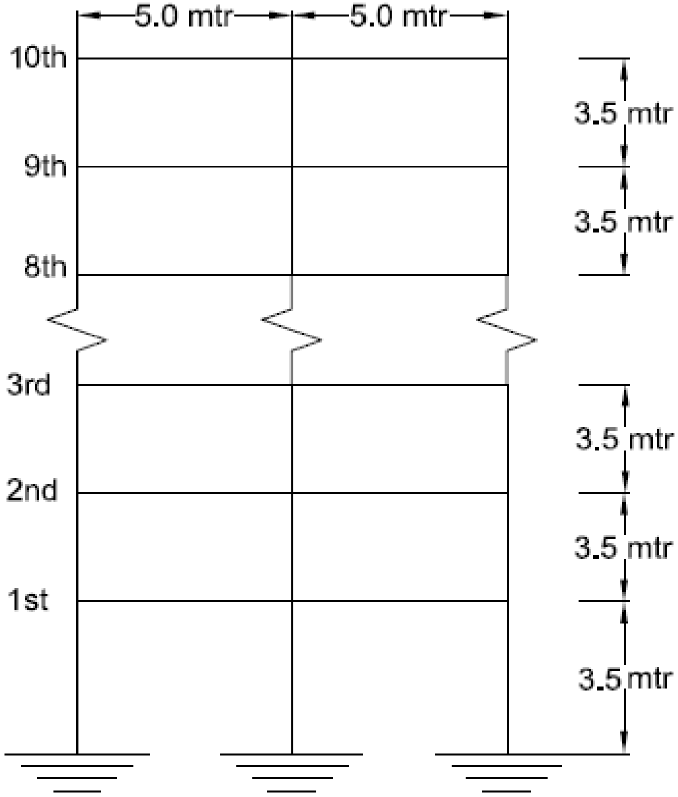


Figure 4.1 (a) Elevation of 2D Plane frame structure.

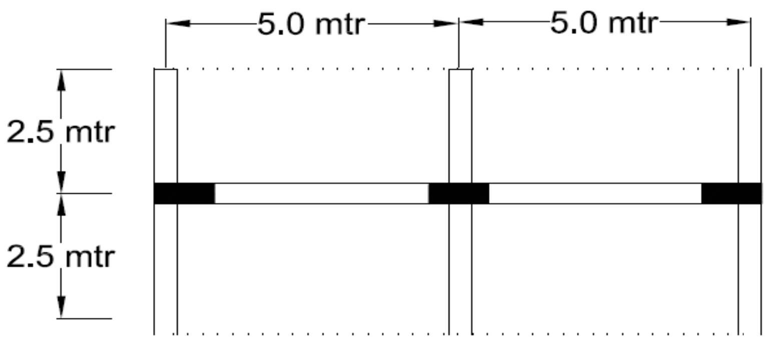


Figure. 4.1 (b) Plan showing the columns and beams at floor levels of the plane frame.

Assumed Preliminary data required for analysis of the frame:

a)	Type of the structure	Multi-storey rigid jointed plane frame.
b)	Number of stories	Ten, (G+9)
c)	Floor Heights & Bay widths	As Shown in figure 4.1
e)	Imposed Load	3.5 kN/m^2
f)	Materials	Concrete (M25) & Steel (Fe415)
g)	Size of columns	250mm x 450 mm
h)	Size of beams	250 mm x 400 mm in longitudinal direction
i)	Depth of slab	100 mm
j)	Specific weight of R.C.C.	25 kN/mm^3

4.2 PRELIMINARY CALCULATIONS:

1. Modulus of Elasticity of Concrete :

   $E = 5000\sqrt{(f_{ck})} = 25{,}000\ N/mm^2$

   $= 2.5x10^{10}\ N/m^2$

2. MOI of Column $I_c = bd^3/12 = 0.25x0.45^3/12$

   $= 1.9x10^{-3}\ m^4$

3. MOI of Beam $I_b = bd^3/12 = 0.25x0.4^3/12$

   $= 1.33x10^{-3}\ m^4$

4. Calculation of Load per unit length.
   a. Loading on Column per unit length.
      i. Self weight of column

         = 0.45x0.25x2500 kg/m

         = 281.25 kg/m

         = 281.25x9.81 N

         = 2759.0625 N

b. Loading on Beam per unit length.

i. Self Load of Beam

= 0.4x0.25x2500 kg/m

= 250 kg/m

= 2452.5 N

ii. Weight of Slab

= 0.1x5.0x2500 kg/m

= 1250 kg/m

= 12262.5 N

iii. Live Load on Slab

= $5.0 \times 3.5 \times 10^3$ N

= 17500 N

Combining, weight per mtr length of beam = 2452.5+12262.5+17500 N

= 32215 N

4.3 FREE VIBRATION ANALYSIS OF THE STRUCTURAL MODEL:

4.3.1 Convergent Study for Natural Frequency:

A convergent study has been carried out to find out the Natural frequencies of the structural model. In total four equivalent models has been considered for this study by changing the number of elements. Number of elements has been increased by introducing equal number of nodes in both beams between each bay and columns between each floor.

Table 4.1 Convergent study for Natural frequencies of the structure (No. of Storey = 10, No. of Bay = 2, Height of each storey = 3.5 m and Width of each Bay = 5 m)

Natural Frequencies (rad/sec)	Number of Elements			
	50	100	150	200
1st Mode	0.8989	0.8033	0.8032	0.8032
2nd Mode	2.7962	2.4851	2.4843	2.4840
3rd Mode	4.9732	4.3997	4.3973	4.3961
4th Mode	7.4286	6.5174	6.5109	6.5076
5th Mode	10.2051	8.9122	8.8993	8.8922

It is observed from Table 4.1 that the fundamental natural frequencies of the structure getting converged for any finer mesh division than 150 elements. Therefore for further study, the structure model considered is 2-D frame model discretized to 150 elements, and the fundamental frequency of the structure is considered as 0.8032 rad/sec. The figure bellow shows the mode shapes for right column of the building.

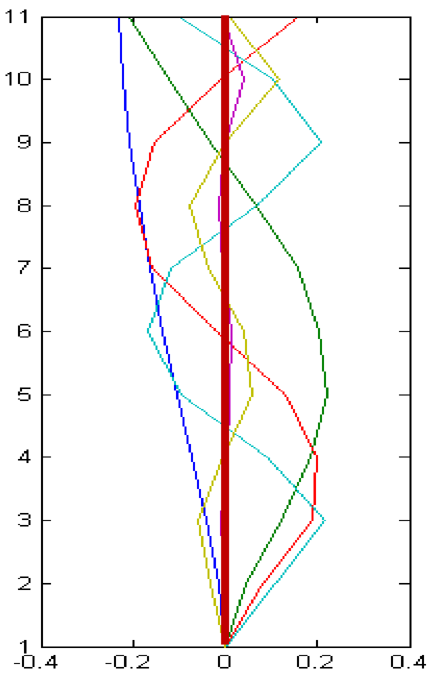


Figure 4.2 First five Mode shapes for the right column of 10 storey structure

4.3.2 Variation of natural frequencies with number of bay:

A study has been carried out to find out the variation of structural natural frequencies, with number of bay. The 2-D frame model has discretized to 150 elements. First five natural frequencies are calculated and compared as produced in table 4.2.

Table 4.2 Variation of Natural frequencies with increase in number of bay (No of Storey = 10, Height of each storey = 3.5 m and Width of each Bay = 5 m)

Natural Frequencies (rad/sec)	**Number of Bays**				
	1	**2**	**3**	**4**	**5**
1st Mode	0.8077	0.8032	0.8008	0.7992	0.7982
2nd Mode	2.5436	2.4843	2.4636	2.4526	2.4458
3rd Mode	4.6346	4.3973	4.3168	4.2750	4.2494
4th Mode	6.9695	6.5109	6.3547	6.2732	6.2233
5th Mode	9.6780	8.8993	8.6318	8.4915	8.4056

It has been observed that there is a slight decrease in natural frequency of the structure with increase in bay numbers. This is because the mass of the structure increases more in proportionate to the increase in stiffness.

4.3.3 Variation of natural frequencies with number of storey:

A study has been carried out to find out the variation of structural natural frequencies, with number of storey. The 2-D frame model has discretized to 150 elements. First five fundamental frequencies are calculated and compared as produced in table 4.3.

Figure. 4.3 Variation of Natural frequencies with increase in number of story (No of Bay = 2, Height of each storey = 3.5 m and Width of each Bay = 5 m)

Natural Frequencies	Number of Storey					
	3	5	8	10	13	15
1st Mode	2.8385	1.6661	1.0180	0.8032	0.6045	0.5156
2nd Mode	9.5260	5.3483	3.1736	2.4843	1.8614	1.5880
3rd Mode	17.3840	9.8838	5.6713	4.3973	3.2840	2.8085
4th Mode	18.9408	15.6713	8.5251	6.5109	4.7873	4.0676
5th Mode	21.4766	17.6427	11.7540	8.8993	6.4522	5.4447

From the table, it is clear that, the fundamental frequency of the structure decreases with increase in number of storey, keeping mass, stiffness of each storey unchanged.

4.4 FORCED VIBRATION ANALYSIS OF THE STRUCTURAL MODEL:

4.4.1 Response of structure to Harmonic Ground Acceleration:

Forced Vibration analysis is carried out for the structure. The structure is subjected to a sinusoidal forced horizontal base acceleration given by:

$$a_x(t) = X_0 \sin(\omega.t) \qquad \text{For } t \geq 0$$

Where, X_0 and ω are the amplitude and frequency of the forced horizontal acceleration respectively. The parameters X_0 and ω are 0.01m and 0.8032 rad/sec (Resonance Condition) respectively. The structure is discretized into 150 elements. The response of the structure at 5th storey and 10th storey are measured in terms of displacement, velocity, acceleration and Base shear as shown in figure 4.3

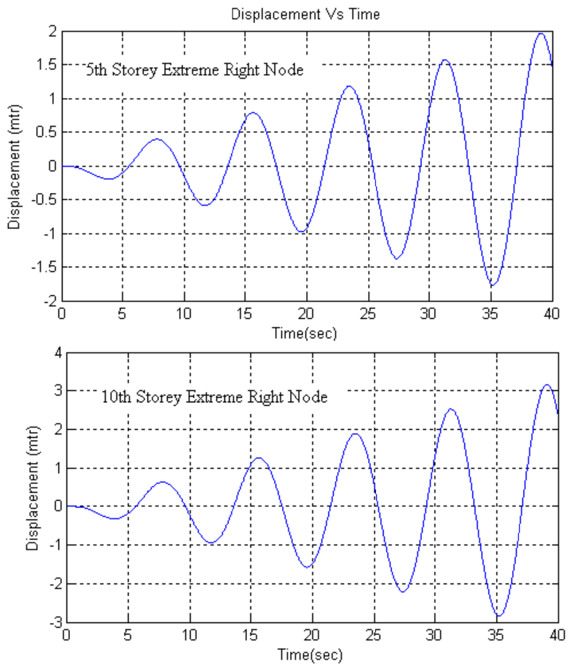


Figure 4.3 (a) Displacement Vs Time

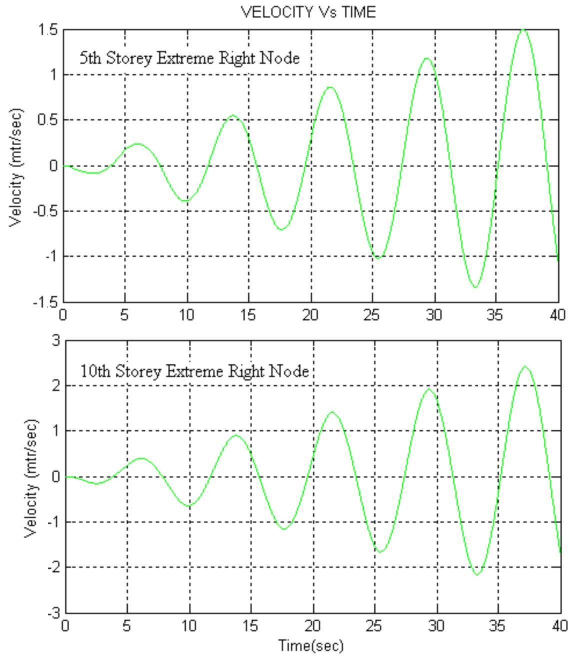


Figure 4.3 (b) Velocity Vs Time

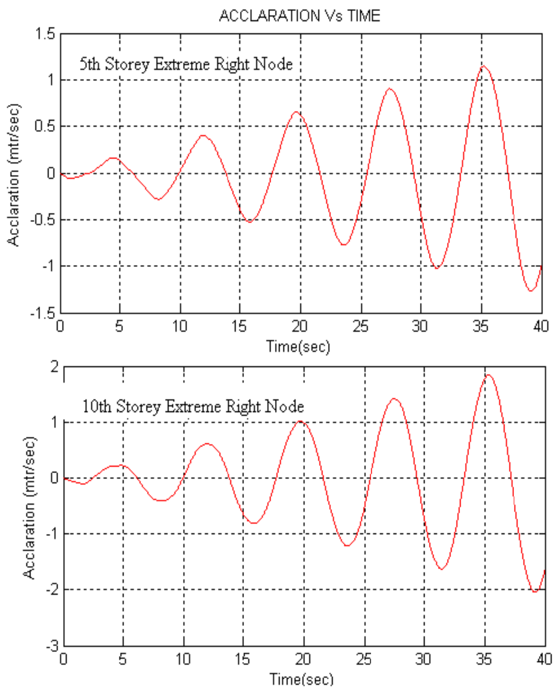


Figure 4.3 (c) Acceleration Vs Time

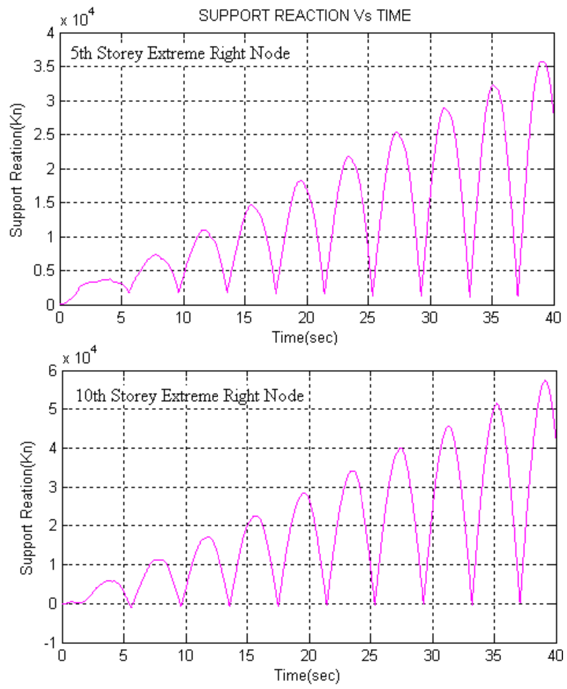
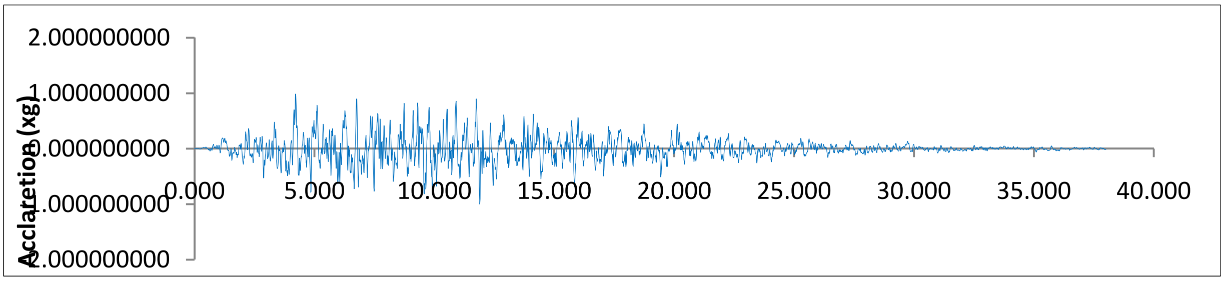


Figure 4.3 (b) Base Shear Vs Time

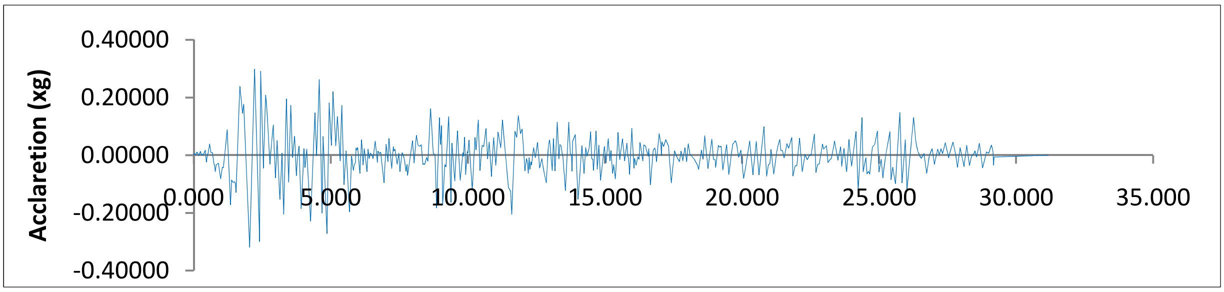
Figure 4.3 Response of 5th and 10th storey of the structure to sinusoidal ground acceleration

4.4.2 Time Histories of Random Ground Acceleration:

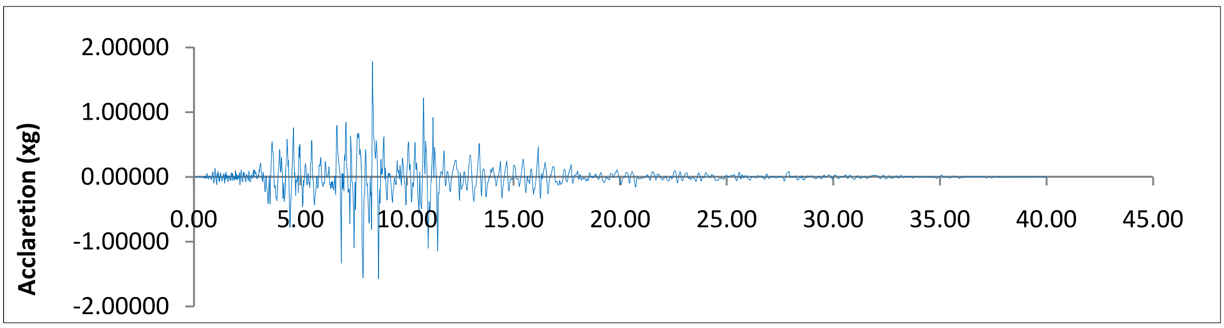
A Total of five random ground acceleration cases are considered for the analysis. First loading chosen is the compatible time history as per spectra of IS-1894 (Part -1):2002 for 5% damping at rocky soil. (PGA = 1.0g) (Figure 4.4 a). Other Four loadings are considered from past earth quake time history data such as 1940 El Centro Earthquake record (PGA = 0.313g) (Figure 4.4 b), 1994 North Ridge Loading (PGA = 1.78g) (Figure 4.4 c), 1971 Sanfernando Earthquake (PGA = 1.23g) (Figure 4.4 d), 1989 Loma Prieta Earthquake (PGA = 0.59g) (Figure 4.4 e).



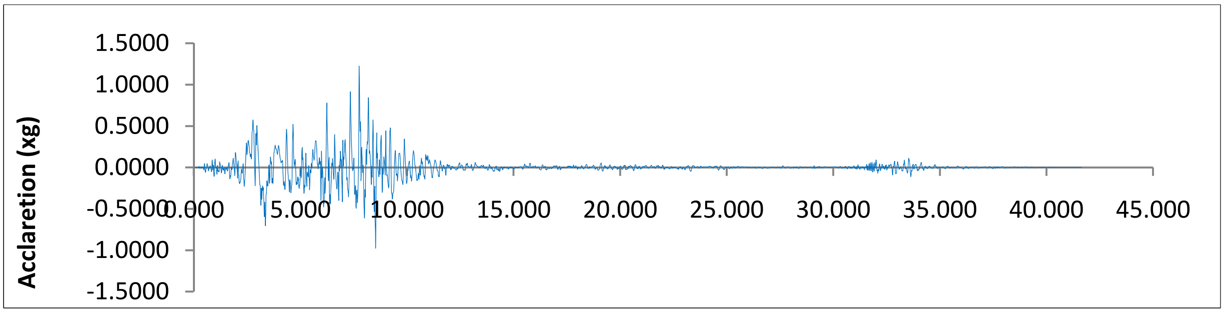
(a) Compatible time history as per spectra of IS-1894 (Part -1):2002 for 5% damping at rocky soil



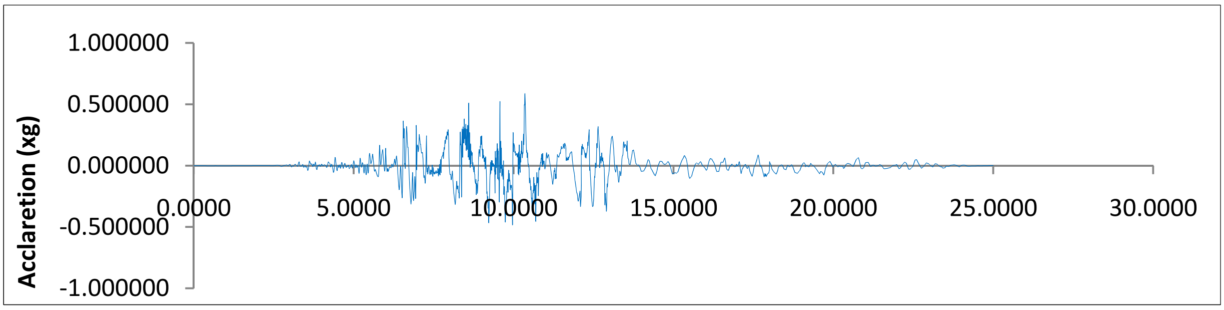
(b) 1940 El Centro EQ Time History



(c) 1994 North Ridge EQ Time History



(d) 1971 Sanfernando EQ Time History



(e) 1989 Loma Prieta EQ Time History

Figure 4.4 Acceleration Time histories of past earth quakes

4.4.3 Response of the structure to Random Ground Acceleration:

The above mentioned time histories are applied on the structure. The response of the structure is measured in terms of amplitude of displacements of right node of 10th storey as shown in the figure 4.5 (a – e)

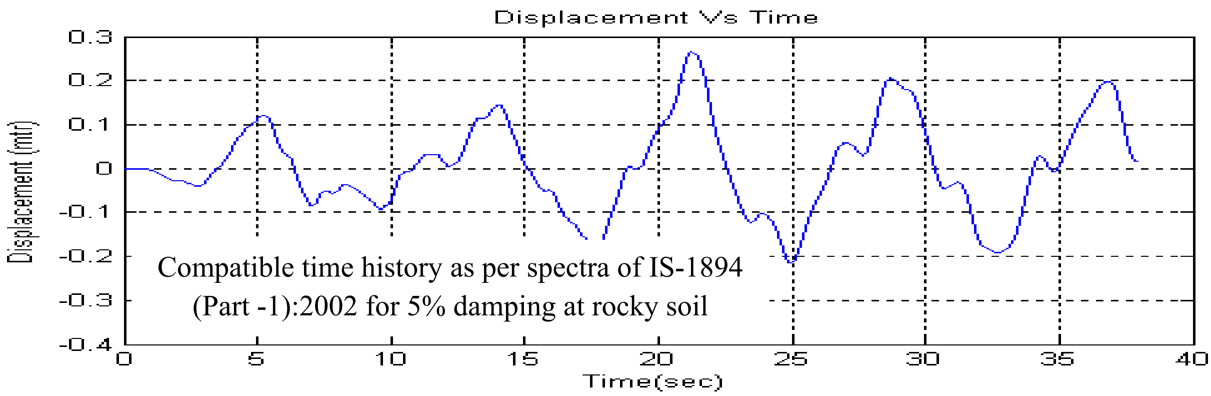


Figure 4.5 (a)

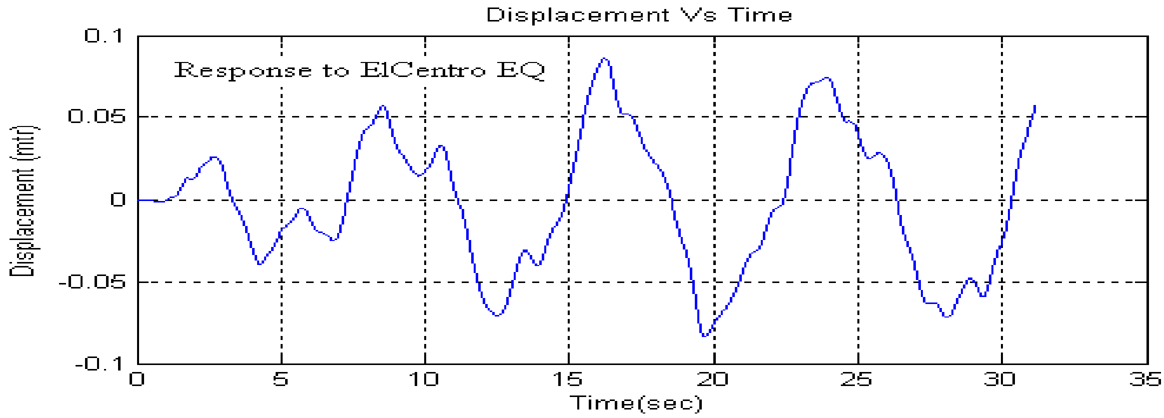


Figure 4.5 (b)

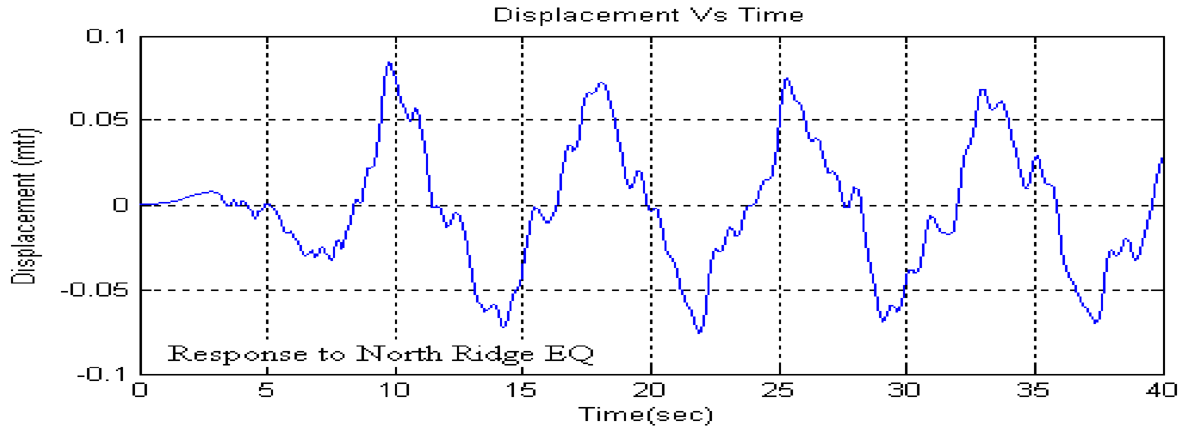


Figure 4.5 (c)

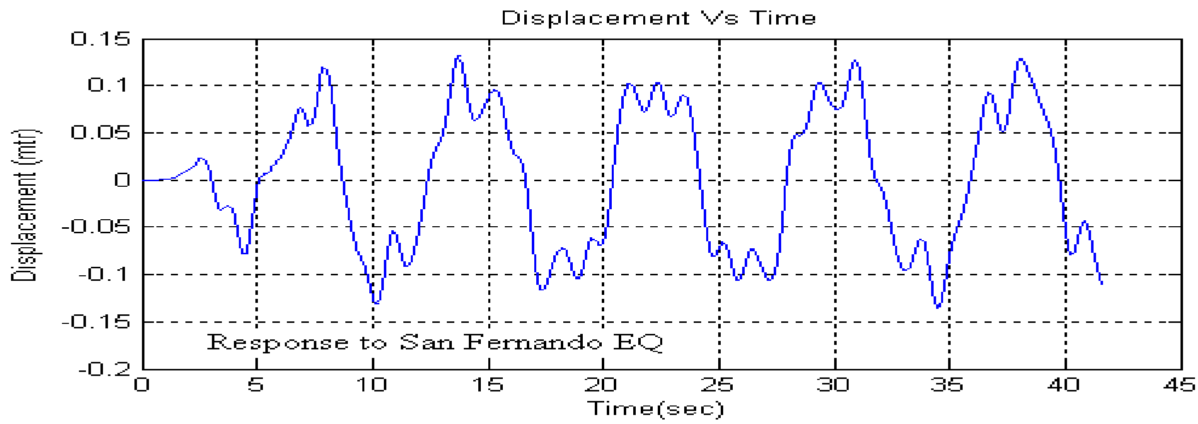


Figure 4.5 (d)

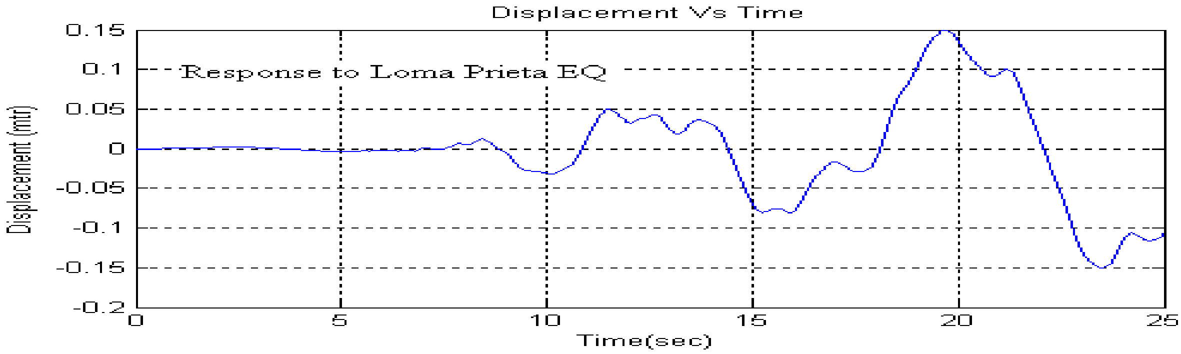


Figure 4.5 (e)

Figure 4.5 Displacement of 10th storey right node to past earthquakes

4.5 DAMPER STRUCTURE ARRANGEMENT:

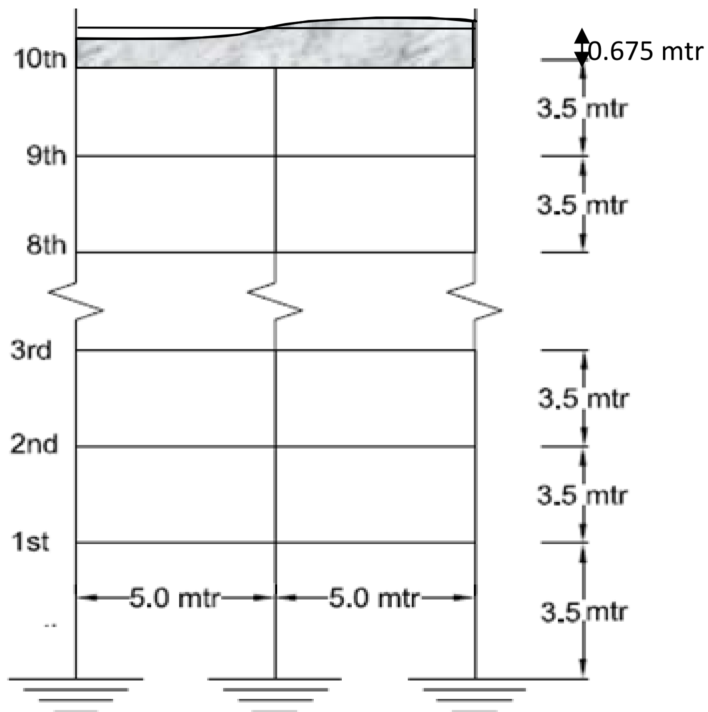


Figure 4.6 Damper-Structure Arrangement

Considering the above arrangement of TLD structure, the length of the Damper considered is 10mtr. The width of the damper considered is 1 mtr. For height of water equal to 0.675 mtr, first natural frequency of water is 0.8024 rad/sec, which is equal to 99.91% of fundamental frequency of structure. Therefore height of water considered is 0.675 mtr. The density of the water (ρ_f) is 1000Kg/m3.

4.6 FREE VIBRATION ANALYSIS OF TLD:

The first six slosh frequencies of liquid are computed and compared with the analytical results reported in NASA monograph edited by Abramson [69].

An analytical expression for the sloshing frequency of liquid in a two dimensional prismatic container without submerged block is given in NASA monograph (SP-106) edited by Abramson [69] and later revised by F.T. Dodge [70] is:

$$\omega_n^2 = \pi(2n-1)\left(\frac{g}{L}\right)\tanh\left[\pi(2n-1)\left(\frac{H}{L}\right)\right]$$ For anti-symmetric modes

and,

$$\omega_n^2 = 2n\pi\left(\frac{g}{L}\right)\tanh\left[2\pi n\left(\frac{H}{L}\right)\right]$$ For symmetric modes

Where, n is the mode number and L is the length of the container respectively and H is the liquid depth and g is the acceleration due to gravity.

Table 4.4, Slosh natural frequencies, f_n (Hz) of liquid in a Rectangular container (L = 10mtr, and H = 0.675 mtr.)

Mode	SP-106 Abramson, 1966	Present Analysis
f1	0.8024	0.8049
f2	1.5710	1.5939
f3	2.2801	2.3537
f4	2.9168	3.0719
f5	3.4797	3.7420
f6	3.9749	4.3550

4.7 FORCED VIBRATION ANALYSIS OF TLD:

4.7.1 Response of TLD to Harmonic Excitation:

Force vibration analysis is carried out for the liquid model. The liquid is subjected to a sinusoidal forced horizontal base acceleration given by $a_x(t) = X_0 \sin(\omega.t)$. Two sinusoidal loading conditions are checked, firstly considering resonance condition by taking X_0 and ω as 0.1 m and 0.1277 Hz and secondly 85% tuning condition by taking X_0 and ω as 0.1 m and 0.1085 Hz.

The parameters X_0 taken as 0.1m. The Liquid model is discretized in 10x10 mess division. The responses of the liquid to these loadings are measured interns of the surface wave amplitude at the free surface on the right wall of the container is considered. figure 4.7

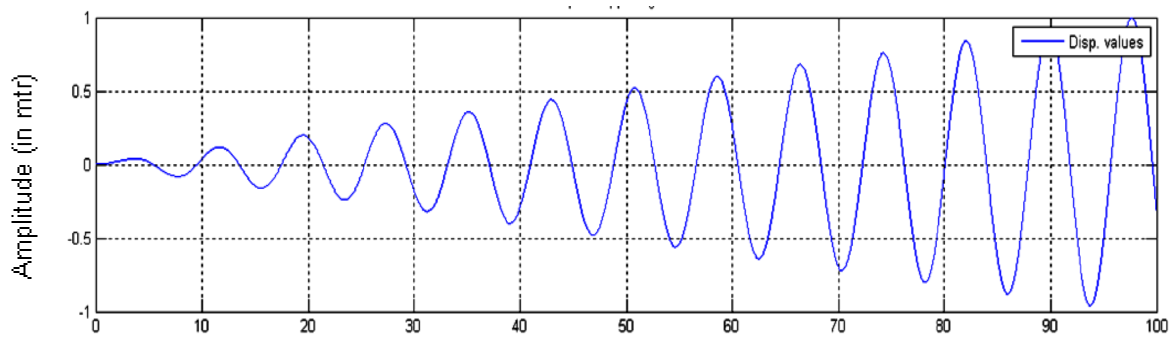


Figure 4.7 (a) (L = 10mtr, H = 0.675mtr, X_0 = 0.1m, ω = 0.1277 Hz)

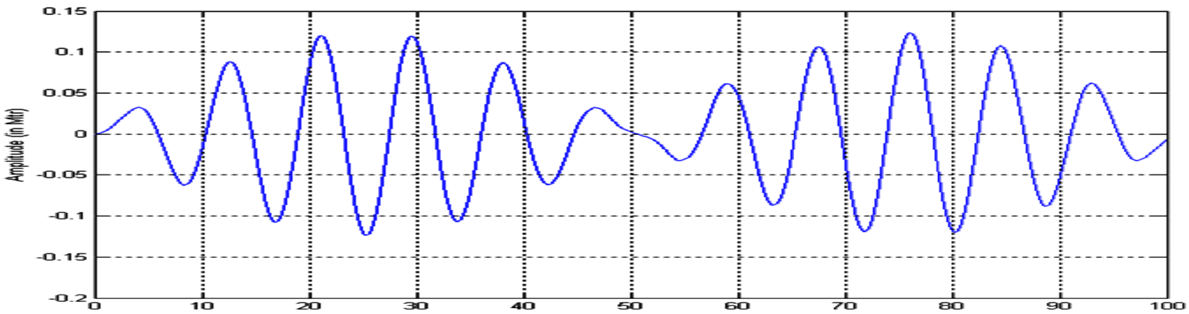


Figure 4.7 (b) (L = 10mtr, H = 0.675mtr, X_0 = 0.1m, ω = 0.1085 Hz)

Figure 4.7 Amplitude of surface wave at the free surface on the right wall of the container

4.7.2 Response of TLD to Random Excitation:

In order to verify the Liquid model forced vibration analysis is carried out for a rectangular container having length (L) of 10.0m and liquid depth (D) of 5.0m. The EW component of the 1940 El Centro Earthquake record (Figure 4.8) with 0.02 seconds intervals is taken as input for the seismic analysis. Here, the earthquake records are scaled with peak acceleration of 0.2g and duration of 60 seconds for easy comparison of the response.

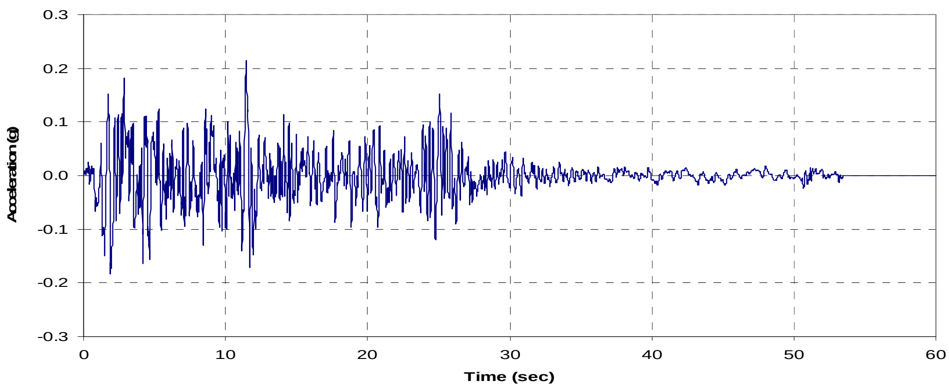


Figure 4.8 The Ground motion for the EW component of the El Centro Earthquake

The sloshing response of the liquid inside the container without any submerged structural component is studied for the EW Component of El Centro ground motion and presented in Figure 4.9 (b) in the form of transient slosh wave. The surface wave amplitude at the free surface on the left wall of the container is considered and compared with the previous results obtained by Choun & Yun (1999) as reproduced in Figure 4.9 (a).

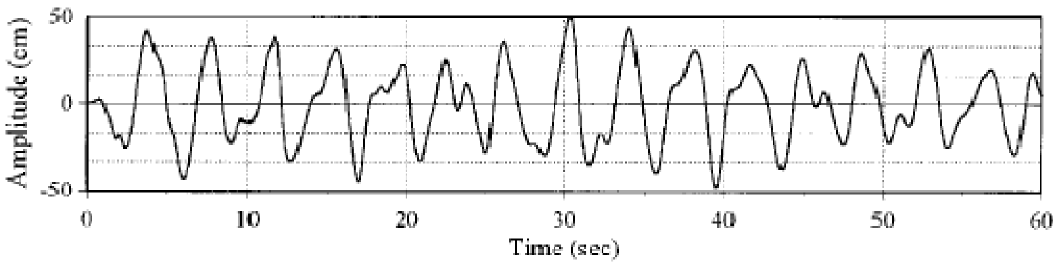


Figure 4.9 (a) Time histories of surface wave amplitude for without submerged component as per Choun & Yun (1999)

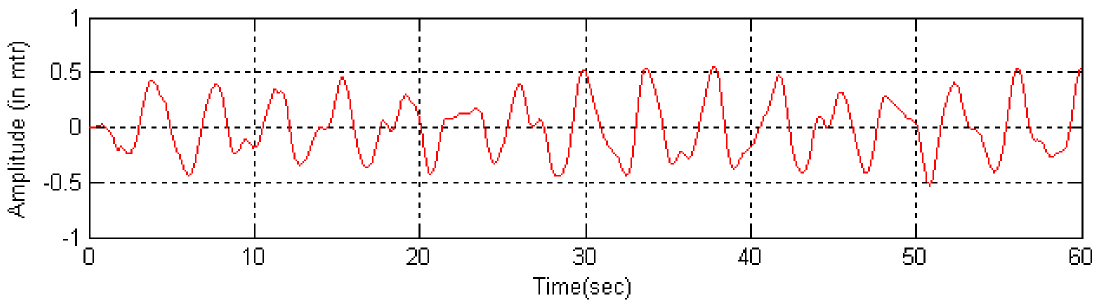


Figure 4.9 (b) free surface displacements on left wall of the container without submerged component as per the present analysis

The same problem is extended to present TLD model, with length of container 10 mtr and height of liquid in container 0.675 mtr. Five Random Ground Accelerations, which have been discussed in article 4.4.2, are applied on the liquid model. The response of the Liquid is measured in terms of interns of the surface wave amplitude at the free surface on the right wall of the container. The results are produced in Figure 4.10.

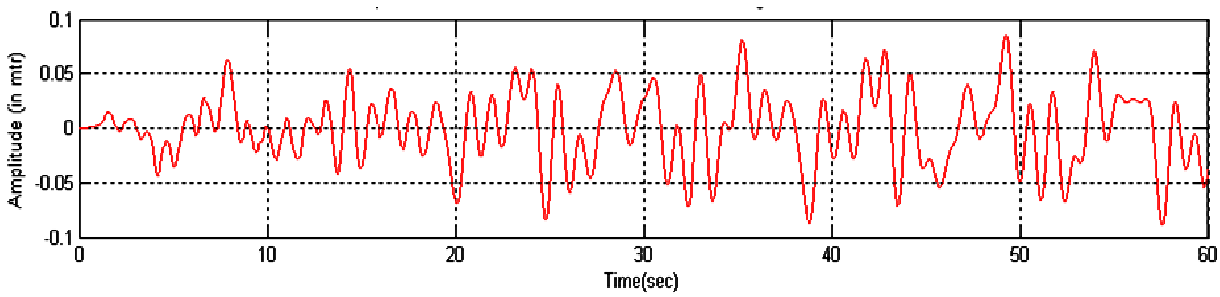


Figure 4.10 Amplitude corresponding to Compatible time history as per spectra of IS-1894 (Part -1):2002 for 5% damping at rocky soil

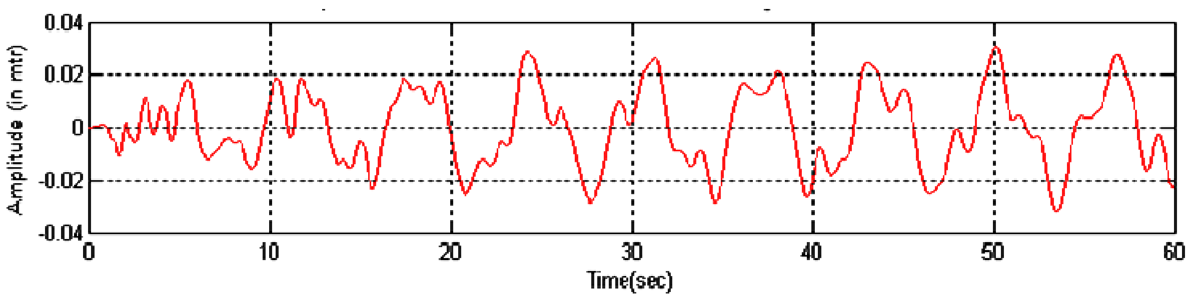


Figure 4.10 (b) Amplitude corresponding to 1940 El Centro Earthquake

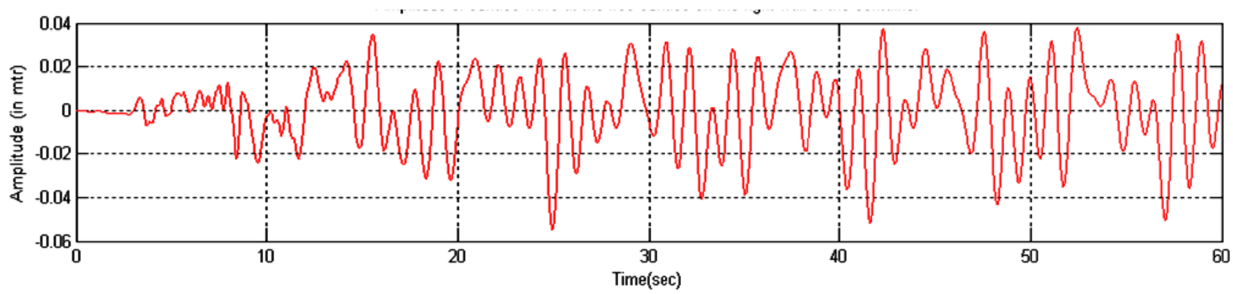


Figure 4.10 (c) 1994 North Ridge Earthquake

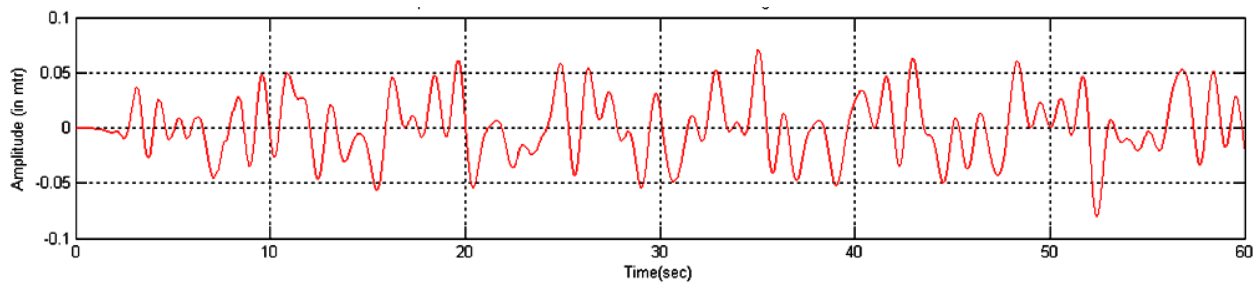


Figure 4.10 (d) 1971 Sanfernando Earthquake

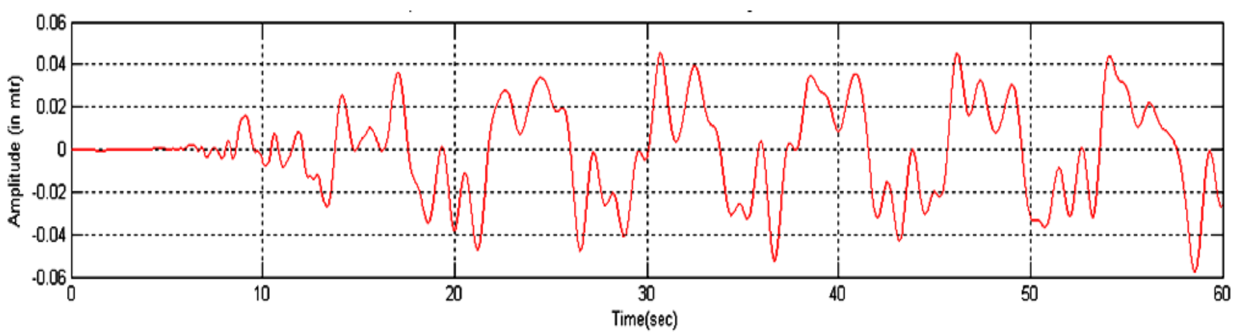


Figure 4.10 (e) 1989 Loma Prieta Earthquake

Figure 4.10 Amplitude of surface wave at the free surface on the right wall of the container

4.8 TLD-Structure Interaction:

4.8.1 Effect of TLD in structural damping when placed at various floors:

To study the effect of TLD on damping of the structure when they are placed at various floors, three cases are considered, by varying the position of TLD at 5th floor, 8th floor and 10th floor respectively. The structure is subjected to a sinusoidal forced horizontal base acceleration given by $a_x(t) = X_0 \sin(\omega.t)$ where X_0 and ω are taken as 0.1 m and 0.8032 rad/sec. The structure is discretized into 150 elements. The response of the structure at 10th storey is measured in terms of amplitude of displacement with TLD and Displacement without TLD.

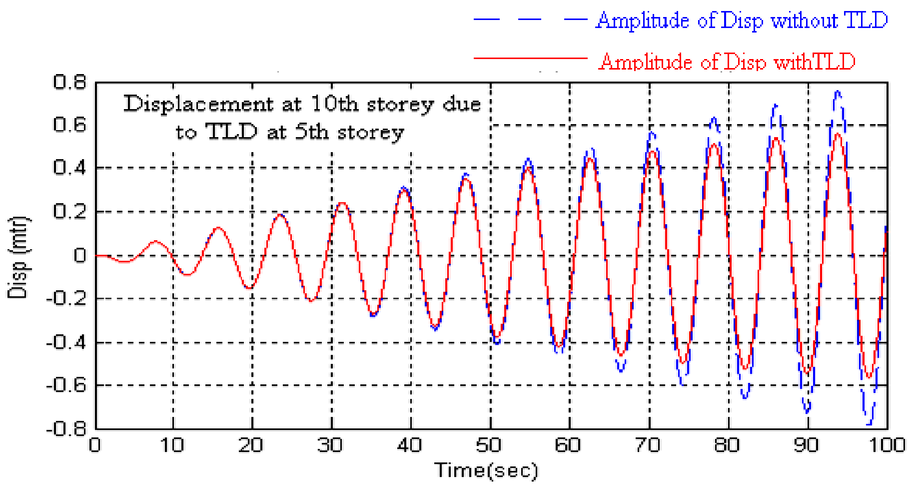


Figure 4.11(a) Displacement of 10th storey due to TLD at 5th storey

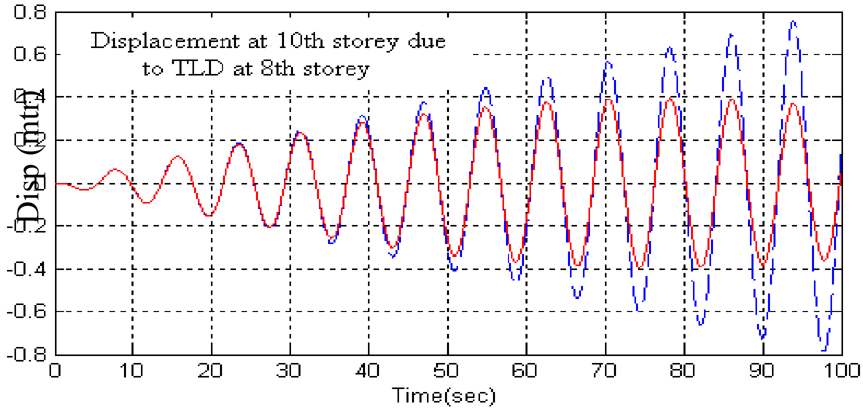


Figure 4.11(b) Displacement of 10th storey due to TLD at 8th storey

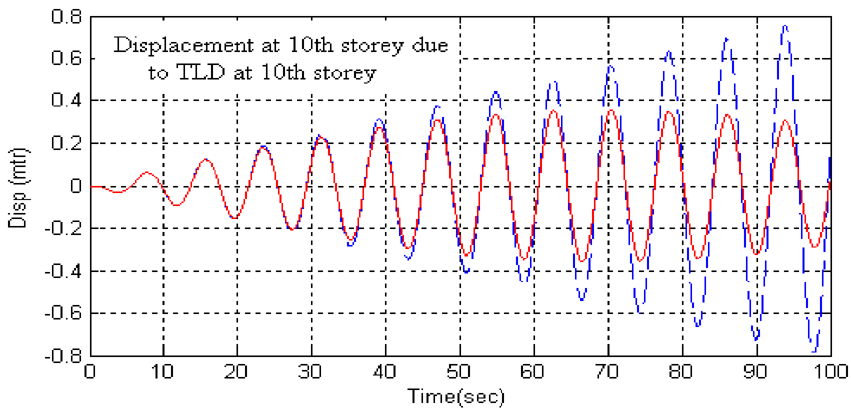


Figure 4.11(c) Displacement of 10th storey due to TLD at 10th storey

Figure 4.11 Displacement at top storey by placing TLD at various floors

From the above figures it has been found that the amplitude of vibration at the top storey is little than 0.6 mtr when we place TLD at the 5th storey, which reduces to 0.4 mtr when we place the TLD at the 8th storey. The amplitude further reduces around 0.35 mtr when we place the TLD at 10^{th} storey. Form the above study it is been found that, the TLD is more effective, when it is placed at top storey.

4.8.2 Effect of Mistuning of the TLD in the damping of the structure:

To study the effect of mistuning of the TLD on damping of the structure, three TLD models are considered. The position of the TLD is fixed at the top of 10^{th} storey of the structure. Mistuning to TLD is made by increasing and decreasing the height of the liquid inside the damper keeping the width and length of the tank constant.

Table 4.5 TLD models considered in study of effect of mistuning of TLD in the damping of the structures

LxH (In mtr)	Total Mass of Water in the Damper (In Kg)	Fundamental Frequency of Damper (F_W) in H_z	Tuning Ratio (T_r) to structural natural frequency
10.0x0.675	6750	0.1277	99.91
10.0x0.61	6100	0.1216	95.10
10.0x0.75	7500	0.1344	105.13

The structure is subjected to a sinusoidal forced horizontal base acceleration given by $a_x(t) = X_0 \sin(\omega.t)$ where, X_0 and ω are 0.1 mtr and 0.8032 rad/sec respectively. The response of the structure at 10th storey is measured in terms of displacement with TLD and Displacement without TLD.

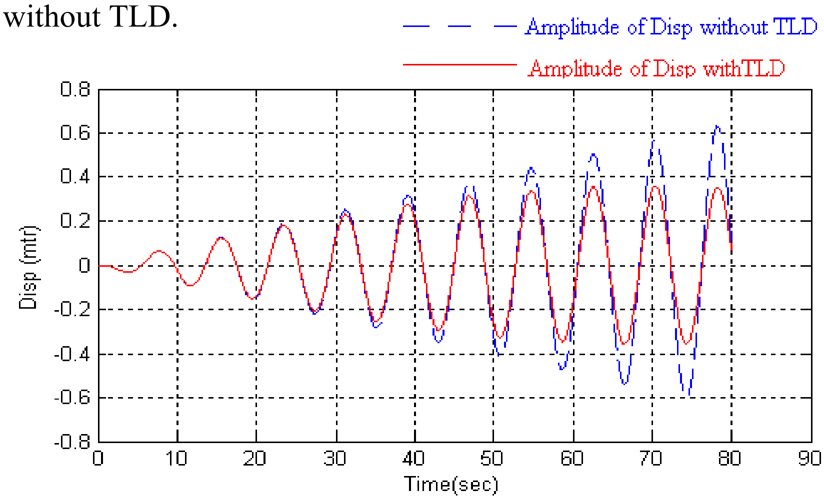


Figure 4.12(a) Response of structure when TLD size is 10.0mX0.675m

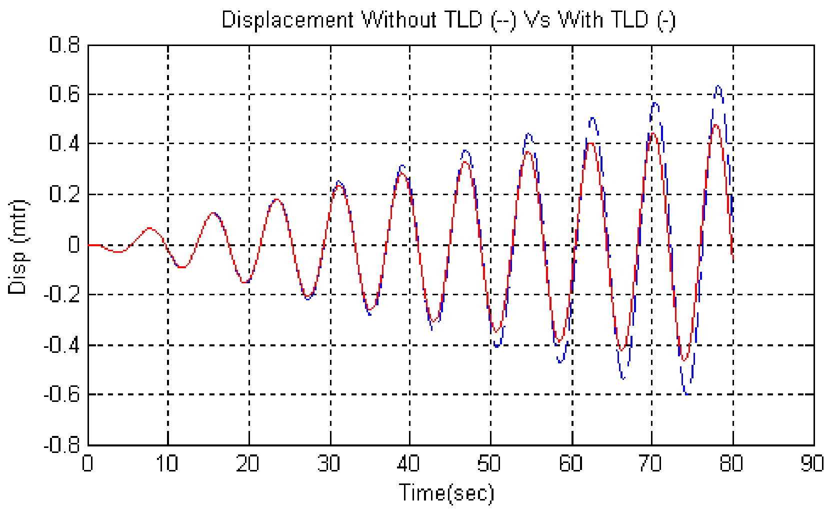


Figure 4.12(b) Response of structure when TLD size is 10.0mX0.61m

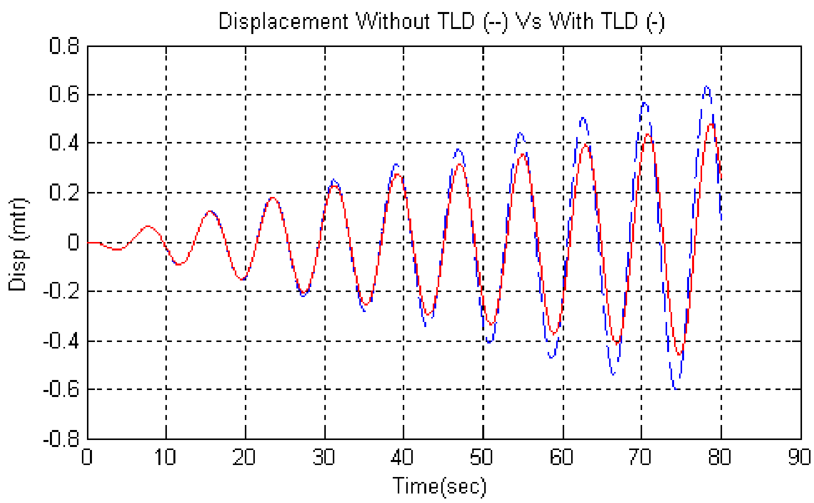


Figure 4.12(c) Response of structure when TLD size is 10.0mX0.75m

Figure 4.12 Displacement at 10^{th} storey due to mistuning of TLD

From the above figures it has been found that the amplitude of vibration at the top storey is around 0.35 mtr when the damper is tuned to natural frequency of the structure. However, when the damper is mistuned slightly, up to around 95% of structural fundamental frequency, the amplitude of vibration at the top storey becomes more than 0.45 mtr, by both under tuning and over tuning the damper.

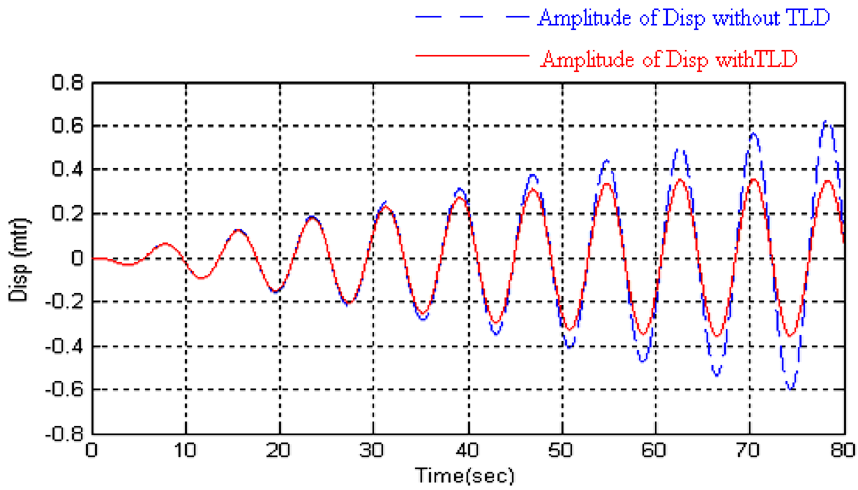
4.8.3 Effect of TLD size in structural damping:

To study the effect of TLD size in structural damping following 3 TLD models of different size are considered. Care is taken to keep the mass of damper nearly equal to 6750 kg. The width of TLD is kept 1mtr in all cases. The position of the TLD is fixed at the top of 10^{th} storey of the structure.

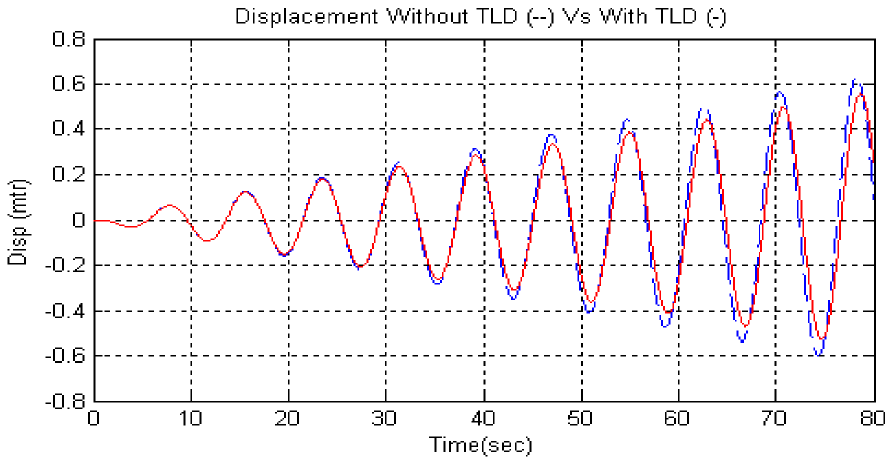
Table 4.6 TLD models considered in study of effect of TLD size in the damping of the structures

LxH (In mtr^2)	Total Mass of Water in the Damper (In Kg)	Fundamental Frequency of Damper (F_w) in H_z	Tuning Ratio (T_r) to structural natural frequency
10.0x0.675	6750	0.1277	99.91
9.5x0.710	6745	0.1376	92.32
9.0x0.750	6750	0.1490	83.44

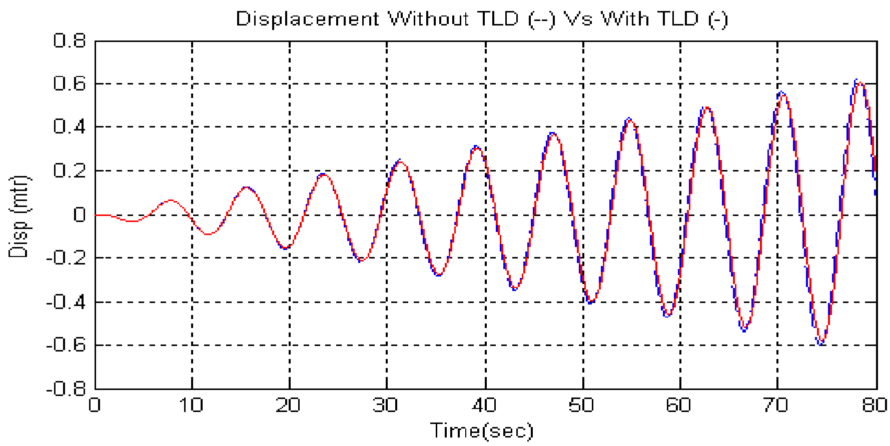
The damper-structure interaction model has been subjected to six loading conditions. First being sinusoidal horizontal base acceleration given by $a_x(t) = X_0 \sin(\omega.t)$ where, X_0 and ω are 0.1 mtr and 0.1277 Hz respectively corresponding to the resonance condition. Five Random Ground Accelerations, which have been discussed in article 4.4.2 are applied at the base of the structure. The response of the structure at 10th storey is measured in terms of amplitude of displacements when TLD is installed as well as amplitude of displacements when TLD is not installed. These responses are placed in the following figures.



a) TLD size is (10 m X 0.675 m)

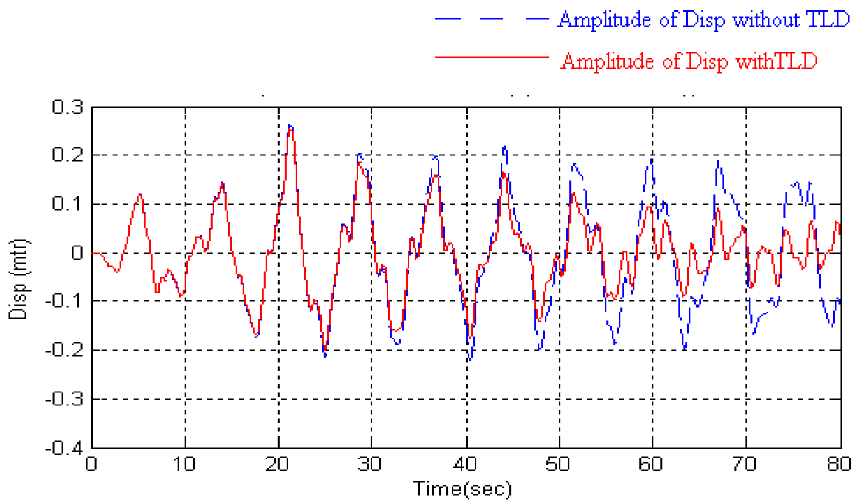


b) TLD size is (9.5 m X 0.710 m)

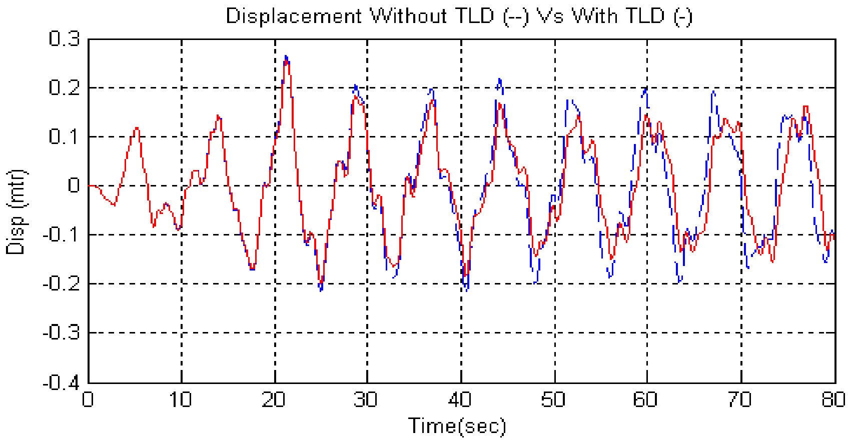


c) TLD size is (9.0 m X 0.750 m)

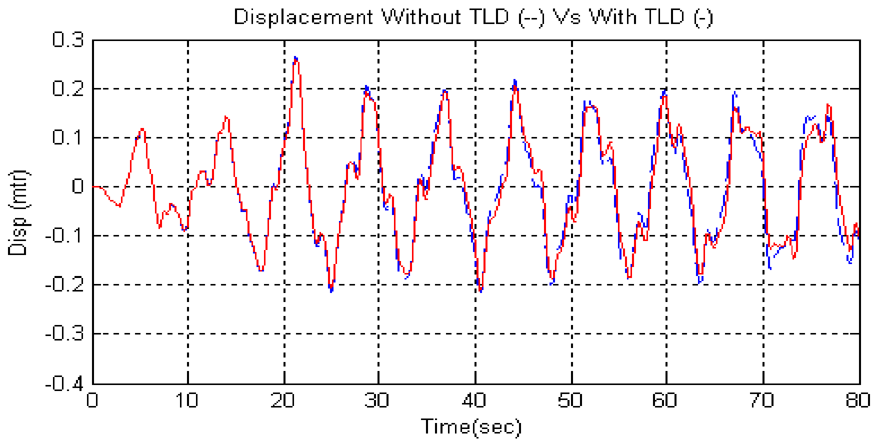
Figure 4.13 Amplitude of vibration at top storey by placing TLD of different size, and when Sinusoidal loading is acting of the structure at resonance condition



a) TLD size is (10 m X 0.675 m)

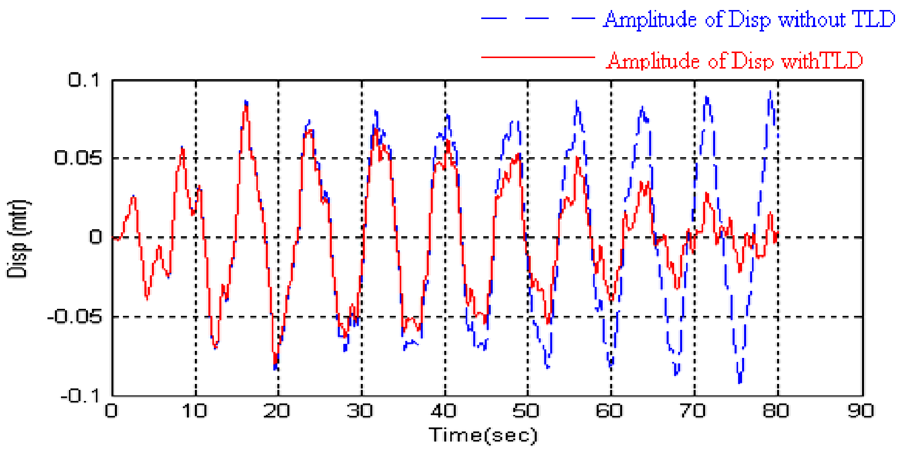


b) TLD size is (9.5 m X 0.710 m)

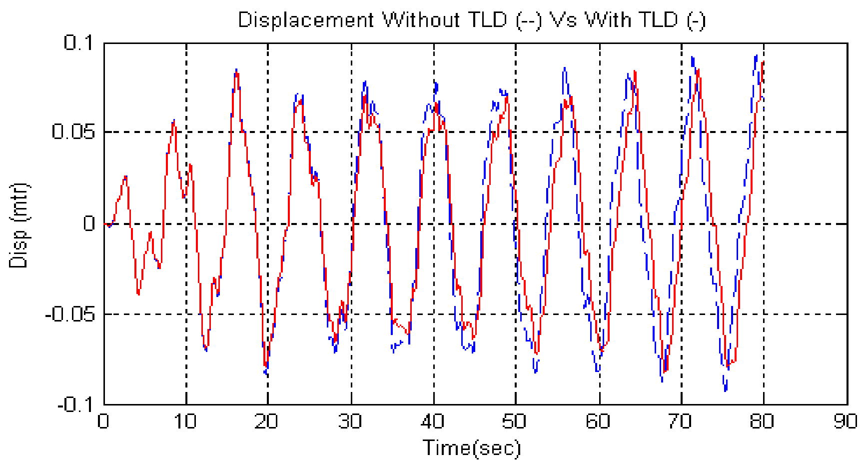


c) TLD size is (9.0 m X 0.750 m)

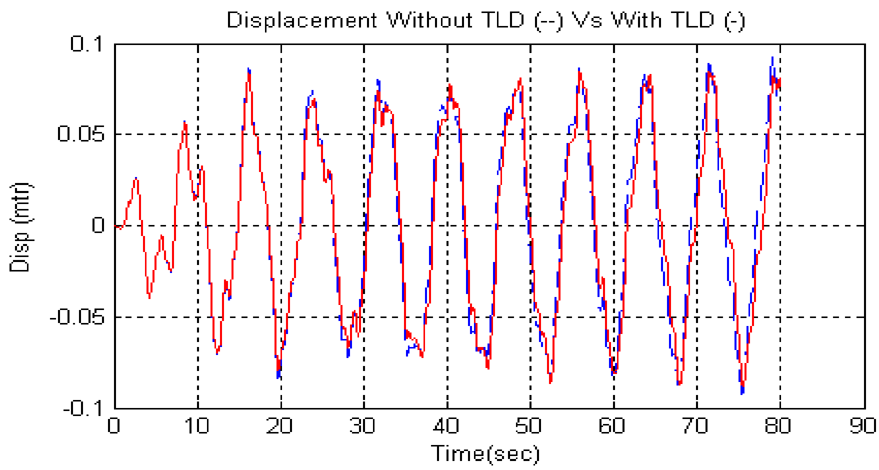
Figure 4.14 Amplitude of vibration at top storey by placing TLD of different size, and when corresponding to compatible time history as per spectra of IS-1894 (Part -1):2002 for 5% damping at rocky soil, acting on the structure



a) TLD size is (10 m X 0.675 m)

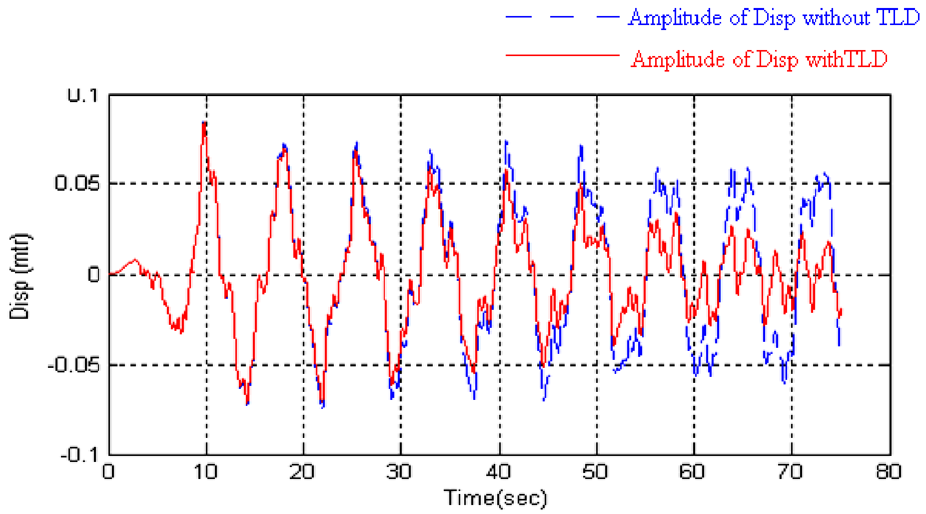


b) TLD size is (9.5 m X 0.710 m)

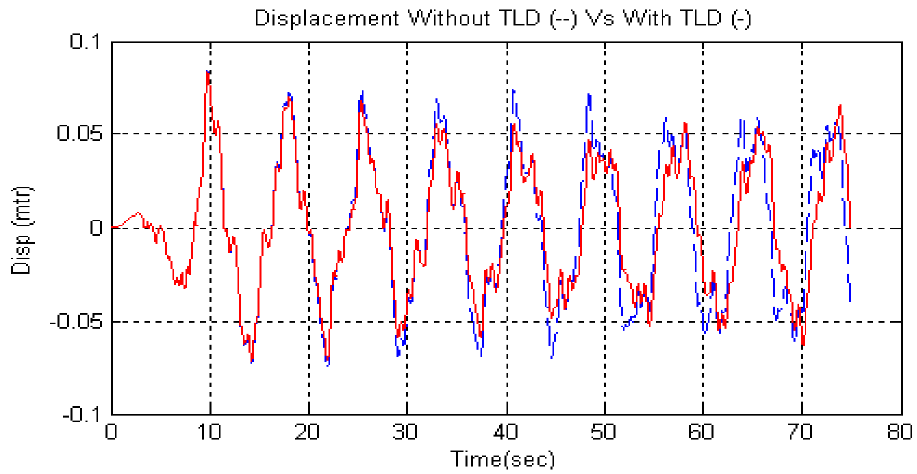


c) TLD size is (9.0 m X 0.750 m)

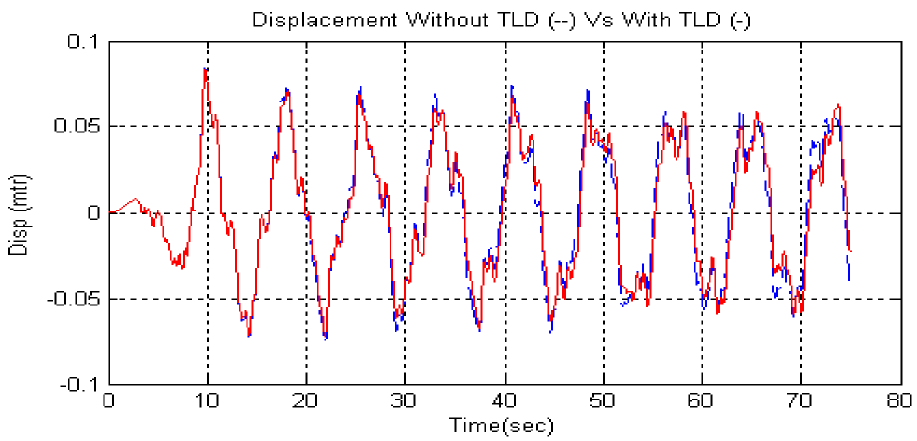
Figure 4.15 Amplitude of vibration at top storey by placing TLD of different size, when El Centro (1940) earthquake load acting on the structure



a) TLD size is (10 m X 0.675 m)

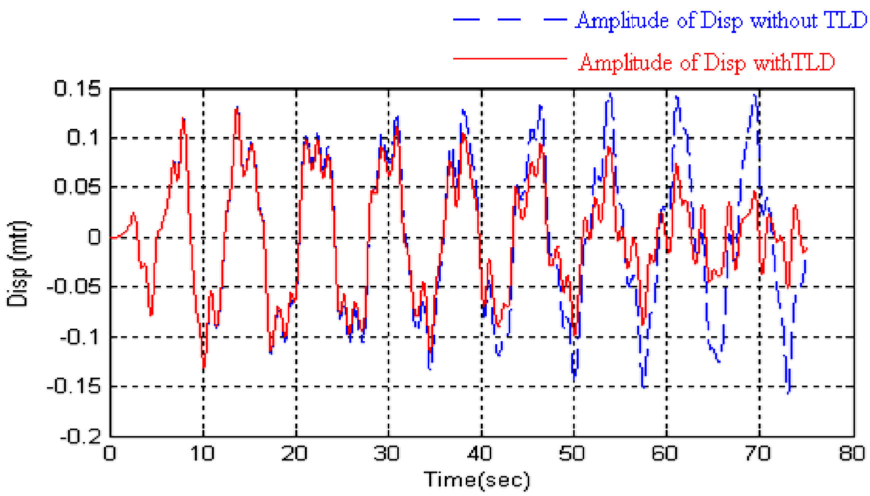


b) TLD size is (9.5 m X 0.710 m)

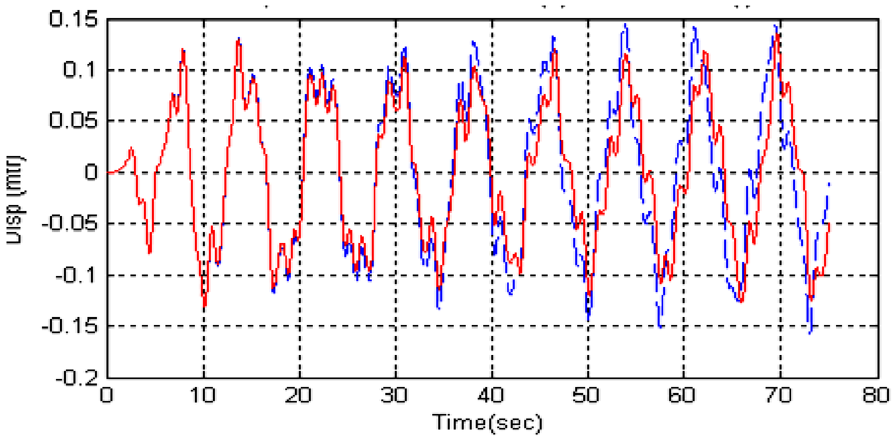


c) TLD size is (9.0 m X 0.750 m)

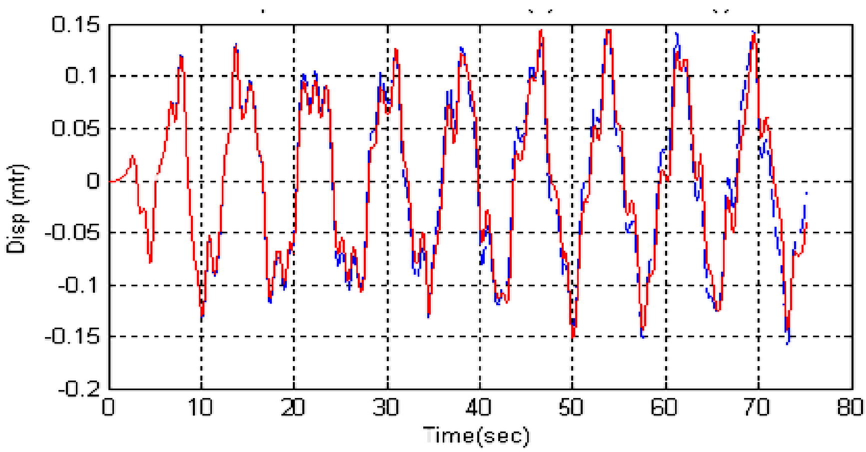
Figure 4.16 Amplitude of vibration at top storey by placing TLD of different size, when North Ridge (1994) earthquake load acting on the structure



a) TLD size is (10 m X 0.675 m)

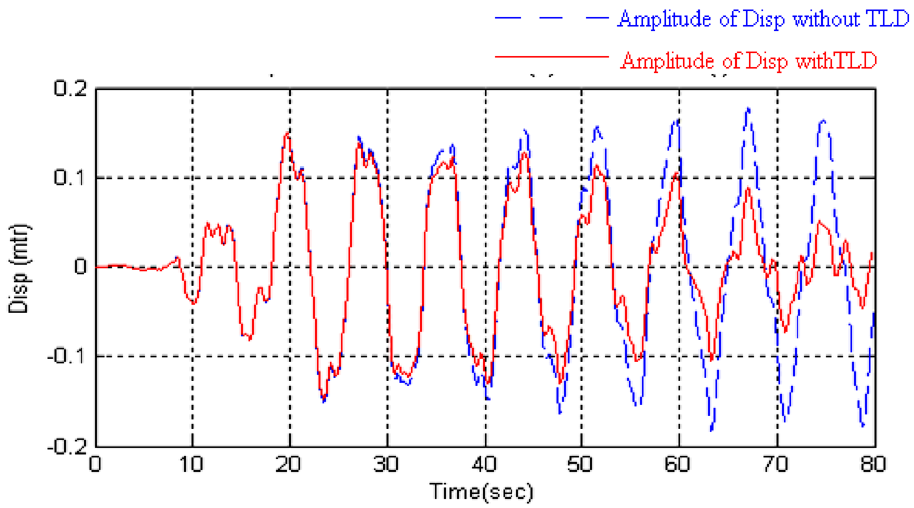


b) TLD size is (9.5 m X 0.710 m)

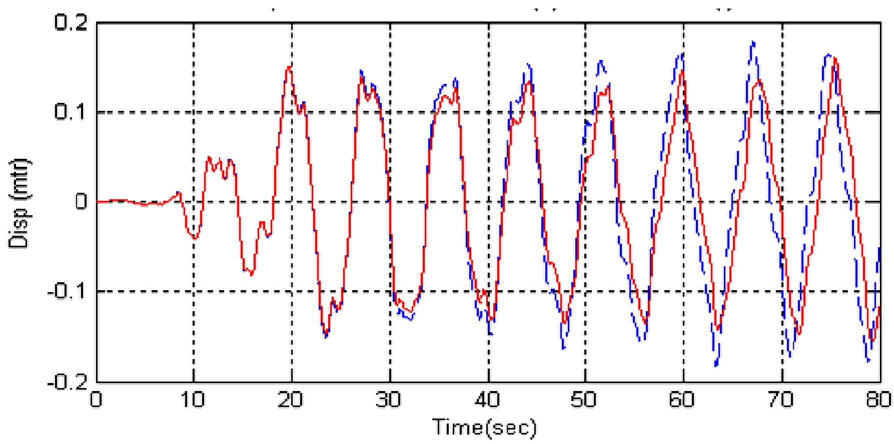


c) TLD size is (9.0 m X 0.750 m)

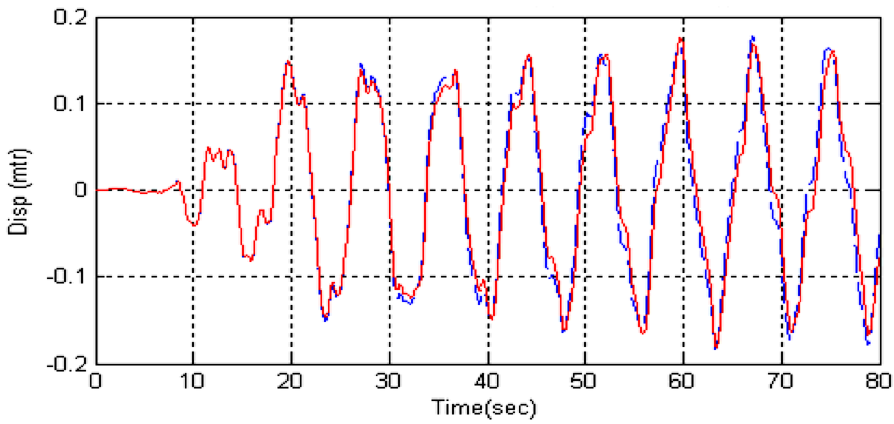
Figure 4.17 Amplitude of vibration at top storey by placing TLD of different size, when San Fernando (1971) earthquake load acting on the structure



a) TLD size is (10 m X 0.675 m)



b) TLD size is (9.5 m X 0.710 m)



c) TLD size is (9.0 m X 0.750 m)

Figure 4.18 Amplitude of vibration at top storey by placing TLD of different size, when Loma Prieta(1989) earthquake load acting on the structure

It has been found, from the figure 4.13 to 4.18 that, the amplitude of vibration at the top storey is decreasing for a TLD size of 10m X 0.675m, whereas no significant decrease in amplitude of vibration of top storey could not seen from graphs of other two damper sizes of 9.5mX0.71m and 9.0mX0.75m. This is because, the first damper is tuned to fundamental frequency of structure; whereas the other dampers are not properly tuned. From this analysis we can conclude, that the TLD to be effective in structural damping, must be tuned properly to fundamental frequency of structure.

CHAPTER - 5 (SUMMARY AND FURTHER SCOPE OF WORK)

5.1 SUMMARY:

Current trends in construction industry demands taller and lighter structures, which are also more flexible and having quite low damping value. This increases failure possibilities and also, problems from serviceability point of view. Several techniques are available today to minimize the vibration of the structure, out of which concept of using of TLD is a newer one. This study is made to study the effectiveness of using TLD for controlling vibration of structure. A numerical algorithm has been developed to investigate the response of the frame model, fitted with a TLD. A linear TLD model is considered. A total of six loading conditions are applied at the base of the structure. First one is a sinusoidal loading corresponding to the resonance condition with the fundamental frequency of the structure, second one is corresponding to compatible time history as per spectra of IS-1894 (Part -1):2002 for 5% damping at rocky soil and rest four are corresponding to time histories of past earthquake such as 1940 El Centro Earthquake record (PGA = 0.313g), 1994 North Ridge Loading (PGA = 1.78g), 1971 Sanfernando Earthquake (PGA = 1.23g), 1989 Loma Prieta Earthquake (PGA = 0.59g). A ten storey and two bay structure is considered for the study. The effectiveness of the TLD is calculated in terms of amplitude of displacements at top storey of the structure.

Following observations and conclusions can be made from this study:

1. From this study, it has been found that the TLD can be successfully used to control vibration of the structure.
2. The TLD is found to be more effective, when it is placed at the top storey of the structure. In the study to access the effect of TLD in structural damping placed at various floors, it has been found that the amplitude of displacement at 10th storey of the structure is 0.35mtr when the TLD is placed at the 10th storey, which increases to 0.6mtr when TLD is placed at 5th floor. The loading applied on the structure is sinusoidal loading at resonance condition to fundamental frequency of the structure.
3. A study has been done to found the effect of mistuning of the damper in damping effect of TLD. It has been found that, TLD is most effective when it is tuned to the fundamental natural frequency of structure. Under tuning or over tuning of TLD to fundamental natural frequency of structure puts adverse

effect on the damping of the TLD. In the study to access the effect of mistuning of TLD in structural damping, it has been found that the amplitude of displacement at 10^{th} storey of the structure is 0.35mtr when the TLD is placed at the 10^{th} storey, which increases to 0.45mtr when TLD is both under tuned and over tuned to 95.10% and 105.13% of fundamental natural frequency of structure respectively.

4. A study has been conducted to find out the effect of TLD size in structural damping while keeping the mass of TLD constant. A total of six loading conditions are considered. First being sinusoidal horizontal base acceleration corresponding to the resonance condition while the next five are random ground accelerations corresponding to past earthquakes. It has been found that the Effect of TLD is significant when TLD is perfectly tuned to the fundamental frequency of the structure, where as the effect is very less while the effect is very less when the TLD is not properly tuned.

5.2 FURTHER SCOPE FOR STUDY:

1. Both the structure and Damper model considered in this study are linear one; this provides a further scope to study this problem using a nonlinear model for liquid as well as for structure.
2. The structure and Damper model considered here is two-dimensional, which can be further studied to include 3-dimensional structure model as well as damper liquid model.
3. Response of Liquid model can be studied by Mess free methods.
4. This study can be done by introducing obstacles like baffles, screens and floating particles, and the change efficiency in the TLD model can be compared.
5. Further scope, also includes studying the possibility of constructing Active TLD using controllable baffles and screens.

CHAPTER-6 (REFERENCES)

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