2. *LITERATURE REVIEW*

2.1 INTRODUCTION

Disasters per se have been dealt by management experts, government and semi-government agencies in the past and the role of engineers has been mostly relegated to retrofitting and strengthening post disaster. Most international codes have now started addressing the situation as disasters are occurring at a higher frequency across the globe. Traditionally declared disaster prone zones are ever expanding into new domains. Thus awareness among engineers, architects and equally among non-engineers has increased and resulted into various alternatives for mitigation and prevention aspects. Available literature examines individual aspects of each disaster. International guidelines describe the methodology that can be adopted for structural analysis and design for earthquakes, but there is no standardized protocol for other disasters. Thus it is difficult to truly handle the complex dynamics of real-time forces of earthquake, wind, fire or flood. Proprietary software on the other hand leave very little scope of flexibility to incorporate specific aspects of forces that could have disastrous effects. Once again the catch in available software is the modeling effort and the assurance of a robust model depicting the real life situation. Thus both literature and software do not enable the engineer to have, on hand, a mechanism of finding a solution to his customized needs nor to study the effects in a post processor instead of tabulated or two-dimensional graphical outputs. Besides, there has been no attempt at mirroring the outputs in a virtual environment which would show the actual behaviour of the building in a real-life manner.

Studies have thus been highly focused on addressing the structural aspects of various disasters. The review of such available literature for the current research is presented here in nine major parts:

- Pushover technique for performance based analysis of buildings.

- Damage and retrofit options for seismic forces
- Virtual reality in structural engineering
- Structural aspects of mitigation of cyclonic wind.
- Fire loads on structures
- Structural alternatives for mitigation of flood effects
- Damage and repair for Blasts and Tsunami

2.2 PUSHOVER ANALYSIS

Codes for structural design have outlined the procedures for static and dynamic analysis of seismic forces and mentioned time-history methods as the final solution. Performance based analysis on the other hand has been effectively used by other disciplines of engineering for detailed designing. Pushover analysis has emerged as a popular tool for seismic analysis using a non-linear static performance based method for evaluating the response of a particular building under a particular force.

Habibullah and Pyle in **1998** [6], presented simple steps for performing pushover analysis using SAP2000 software. The SAP2000 static pushover analysis capabilities, which are fully integrated into the program, allow quick and easy implementation of the pushover procedures prescribed in the ATC-40 and FEMA-273 documents for both two and three-dimensional buildings.

Moghadam and Tso in **2000** [7], extended pushover analysis to cover plan-eccentric buildings and took the three-dimensional torsional effect into account. Because of torsional deformation, floor displacements of the building will consist of both translational and rotational components. Torsional effect can be particularly damaging to elements located at or near the flexible edge of the building where the translational and rotational components of the floor displacement are additive. In view of the damage observed in many eccentric buildings in past earthquakes,

Chopra in **2001** [8] stated in a PEER report that the standard response spectrum analysis (RSA) for elastic buildings is reformulated as a Modal Pushover Analysis (MPA). The peak response of the elastic structure due to its nth vibration mode can be exactly determined by pushover analysis of the structure subjected to lateral forces distributed over the height of the building according to $s_n = m\Phi_n$, where m is the mass matrix and Φ_n its nth-mode, and the structure is pushed to the roof displacement determined from the peak deformation Dn of the nth-mode elastic SDF system. Combining these peak modal responses by modal combination rule leads to the MPA procedure. Thus the trend of comparing computed hinge plastic rotations against rotation limits established in FEMA-273 to judge structural performance should be replaced and Performance evaluation should be based on story drifts known to be closely related to damage and can be estimated to a higher degree of accuracy by pushover analyses.

Another **PEER report** in **2001**[9] given by the same team investigates the basic premise that the roof displacement of a multistory building can be determined from the deformation of an SDF system. The data for generic frames indicate that the first- mode SDF system overestimates the median roof displacement for systems subjected to large ductility demand μ, but underestimates for small μ. The SDF estimate of roof displacement due to individual ground motions can be alarmingly small (as low as 0.312 to 0.817 of the value for the six SAC buildings) or surprisingly large (as large as 1.45 to 2.15 of the value for Seattle and Los Angeles buildings), when P-delta effects were included. The situation was worse than indicated by these data because they do not include several cases where the first mode SDF system collapsed but the building as a whole did not.

Owing to the simplicity of inelastic static pushover analysis compared to inelastic dynamic analysis, the validity and the applicability of this technique were assessed by **Mwafy** in **2001** [10]. Comparison with 'dynamic pushover' idealised envelopes were obtained from incremental dynamic collapse analysis. This involved successive scaling and application of each accelerogram from 12 experiment buildings, followed by assessment of the maximum response, up to the achievement of the structural collapse. The results of over one hundred inelastic dynamic analyses using a detailed 2D modelling approach for each of the twelve RC buildings were utilised to develop the dynamic pushover envelopes and compare these with the static pushover results. Good correlation was obtained between the calculated idealised envelopes of the dynamic analyses and static pushover results for a defined class of structure.

FEMA-368 (2001)[11] define criteria for the design and construction of new buildings, additions and alterations to existing buildings to enable them to resist the effects of earthquake ground motions. These provisions provide minimum seismic design criteria of safety for structures by minimizing the earthquake-related risk to life and improve the capability of existing structures to function during and after design earthquakes. Whereas, **FEMA-369 (2001)** [12] provides general requirements, background information, and explanations for applying the analysis and design criteria in the Provisions of FEMA-368.

Hasan R. in **2002** [13] presented a simple computer-based push-over analysis technique for performance-based design of building frameworks subject to earthquake loading. The technique was based on the conventional displacement method of elastic analysis. Through the use of a plasticity-factor that measured the degree of plastification, the standard elastic and geometric stiffness matrices for frame elements (beams, columns, etc.) were progressively modified to account for nonlinear elastic–plastic behavior under constant gravity loads and incrementally increasing lateral loads. The method accounted for first-order elastic and second order geometric stiffness properties, and the influence that combined stresses have on plastic behaviour.

After designing and detailing the reinforced concrete frame structures, **Korkmaz** in **2002** [14] carried out a nonlinear pushover analysis and nonlinear dynamic time history analysis for evaluating the structural seismic response for the acceptance of load distribution for inelastic behavior. It was assumed for pushover analysis that seismic demands at the target displacement are approximately maximum seismic demands during the earthquake. First yielding and shear failure of the columns was experienced at the larger story displacements and rectangular distribution always give the higher base shear-weight ratio compared to other load distributions for the corresponding story displacement. The pushover analyses results for rectangular load distribution estimated maximum seismic demands during the given earthquakes were more reasonable than the other load distributions.

A trial application of the SAC-FEMA method was presented by **Lupoi** in **2002** [15]. Existing RC buildings not designed for earthquake loads may fail due to several possible weak mechanisms, whose relevance was unpredictable before an accurate analysis was carried out. Concepts and procedures of the SAC-FEMA method were proven to apply to such complicated cases as well. In particular, the stability of the final outcome of the analysis, the total annual risk, record-to record variability, randomness of the material properties and inadequate knowledge on the capacity side, was fully confirmed. This fundamental feature, together with the relative simplicity of the approach, made it all the more desirable for it to be gradually adopted as a design method for new, well conceived and detailed, earthquake resistant constructions

Menjivar and Pinho in **2003** [16] extended the pushover method to asses the performance of 3D irregular RC structures. The issues of diaphragm effects, loading profiles and incremental dynamic analysis were studied. The modeling based on Displacement Based and Force Based Pushover was compared. Conventional verses Adaptive Pushover results have been compared and were found to be close.

Jan in **2004** [17] stated that when evaluating the seismic demands of tall buildings, engineers were more likely to adopt simplified non-linear static analytical procedures, or pushover analyses, instead of the more complicated non-linear response history analysis. Since the conventional procedure has some drawbacks in predicting the inelastic seismic demands of high-rise buildings, in this paper, a new simplified pushover analysis procedure, considering higher mode effects, was proposed. The basic features of the proposed procedure were the response spectrum-based higher mode displacement contribution ratios, a new formula for determining the lateral load pattern and the upper-bound (absolute sum) modal combination rule for determining the target roof displacement.

2.3 DAMAGE AND RETROFITTING FOR EARTHQUAKES

The performance level of buildings is decided based on building use requirements. From the demand–capacity spectra, if the performance point does not fit the required level, it becomes clear that the damage occurring to structural elements needs to be addressed. Based on the type and extent of damage that has occurred retrofitting measures are taken,. Thus it is important that the type of damage be understood by the designer and then the appropriate retrofit strategy may be decided. The following section examines different types of damages and retrofit strategies advocated in technical literature.

Paulay and Priestly (1991) [18], in their book have described various failure modes, responses, ductile design, determination of design forces and concept of capacity design in detail.

FEMA-274 (1992) [19] provides commentary on the NEHRP guidelines for the seismic rehabilitation of buildings. It should be used in connection with FEMA-356, Prestandard and Commentary for the Seismic Rehabilitation of Buildings. Techniques for seismic rehabilitation of existing buildings and the relative merits of the techniques are discussed in **FEMA-172 (1992)** [20].

Various schemes for strengthening deficient RC building frames were discussed by **Jain** [21] in **1993**. Reasons for seismic deficiency, seismic strengthening procedures by either decreasing the seismic force or increasing seismic capacity of building have been described along with a case history of a low rise building in zone III has been given

Boroschek in **2000** [22] conducted studies of structural retrofitting of various facilities and had defined the entire process in simple steps from Preliminary Vulnerability study, Selection of Performance objectives, Site characteristics, Building characteristics, Selection of retrofit procedures and Analysis techniques.

Seismic retrofitting by conventional methods was examined by **Sheth** in **2002** [23]. The author has discussed different methods in light of the predominant failure patterns of various structural systems and different parameters that govern the choice of retrofit. Retrofitting an existing structure needs more experience and judgment as compared to constructing a new structure for seismic resistance. Basic issues such as retrofit evaluation, steps for retrofitting, level of upgradation, retrofit strategies and conventional methods have been discussed in order to introduce ductility in the existing structure. Introduction of new lateral force resisting elements have also been examined.

Also in 2002 **D'Ayala and Charleson** [24] reviewed seismic strengthening guidelines in developing countries, after major earthquakes in Turkey and India. Seismic strengthening guidelines from four major areas of India, USA, Europe and New Zealand were detailed for existing structures.The emphasis was on construction quality assurance, importance of detailing and arriving at a checklist based on the FEMA system for evaluation.

Kotsovos in **2003** [25] investigated causes of unexpected damage suffered by structural elements during the July 1999 Athens earthquake. Elements designed using the truss analogy method as described in most codes, hadundergone brittle failure under monotonic and cyclic loading due to destruction of bond in steel and concrete leading to premature reduction in strength.

Agarwal in **2003** [26] compiled a series on identification of seismic damages in RC buildings in Bhuj, highlighting damages due to soft storey failure, floating columns, plan and mass irregularity and quality of construction material. Damage to various structural elements, infill walls, exterior walls, water tanks and parapets, staircases and elevators were studied in detail. Effect of earthquake on code designed structures was also examined. Effect of structural irregularities of strength, stiffness, mass, geometry and plan, on the performance of RC buildings during earthquakes was examined and recommendations for same were stated.

IITK-GSDMA-EQ03-V1.0 (2003) [27] comparedkey concepts and how they are handled by various international agencies. The major among these are configuration related checks, strength related checks, global and component level checks.

Shake Table tests on 3D RC frames were carried out by **Cardone** et al in **2004** [28] to evaluate the effectiveness of passive control bracing systems for seismic retrofit of frames designed for gravity loads only. The retrofitted model could take three times accelerations leading the unprotected model to collapse, with no significant damage to elements. The re-centering capability of the Shape Memory Alloy (SMA) based bracing system was able to recover the undeformed shape of the frame, when it was in the near collapse condition.

2.4 VIRTUAL REALITY IN STRUCTURAL ENGINEERING

Analysis outputs have traditionally been given in the form of tabulated results or as 2-D graphics. Thus the engineer has to depend on his visualization capabilities to judge the behaviour under a particular loading aspect. Virtual Reality provides an efficient tool for studying the effects of seismic and wind forces under different parameters. Research in this area is limited by the cost involved in the immersed virtual reality, though more and more international universities are showing interest, as seen in the literature presented.

Impelluso in **1995** [29] presented one of the first attempts to create a "Physically-Based Virtual Reality". By linking a linear three-dimensional finite element program with a visualization application, he could present the small deformations of virtual objects subjected to various loading conditions. This effort has later been extended to include the non-linear behavior of objects.

Ardino et al in **1997** [30] created a virtual geotechnical triaxial laboratory in order to model the stress-strain behaviour of soil specimens. Soil properties and confinement levels were controlled by the user. The mechanical response of the virtual soil specimen was displayed in a graphical format using the concept of classroom blackboard.

Camara et al in **1998** [31] studied water resources using multimedia and virtual reality. The virtual reality water quality management system represented a water shed in 3 dimensions and graphical water quality models. The user may fly, walk and dive in the estuarine basin, manipulate the infrastructure system including the polluters and visualize the results in real time. The interface of the system follows video game guidelines. Some illustrative applications of a prototype in a game like environment were successfully used by many users.

Demsetz in **1998** [32] stated the experiences in controlling a multi-robot-system.The general aim of the development was to provide the framework for "*Projective Virtual Reality*" which allows the projection of actions that were carried out by users in the virtual world into the real world with the help of robots. In the virtual simulation, these robots can walk forward, backward and can turn; they can handle tools with their hands, and they can even stoop to pick something up from the floor.

Murakami et al(1998) [33] developed a Virtual Reality (VR) based computer aided design system for Tensegrity Structures. The VR-based CAD system allowed the user to interact with virtual structures through joysticks which apply reaction forces computed by the finite element code. Tensegrity is a class of truss structures which consists of tendons (pure tension members) and bars (compression members), A physically-based virtual reality (VR) system, in which a real-time finite element (FE) code was employed to synthesize a physical world involving deforming objects, was first developed by Impelluso (1995). By modifying Impelluso's system to include tensegrity structures, a VR-based CAD system was developed. In the system, a high performance workstation executed an FE code and displayed a virtual world in 3D by using either head mounted display (HMD) or CrystalEyes. The deformation of the two-stage tensegrity structure was induced each time the operator moves the joysticks. The operator "feels" reaction forces as s/he deforms the virtual structure by using the joysticks. At the same time, the operator can walk-through the deforming structure to investigate deflection and stress states.

Beier in **2000** [34] used VR on the world wide web through VRML modeling. A VRML model is defined in a VRML file which is a regular text file that describes a virtual model using a standardized syntax. The content of a VRML file can be viewed interactively through the use of a VRML plug-in available for all common Web browsers. Using VRML on the World Wide Web provides an excellent tool for sharing virtual models with remote users and for supporting collaborative work and concurrent engineering. It is extremely cost effective since the required infrastructure (networked computers) exists almost everywhere and the viewing software (VRML plug-in) is available to everyone. The Virtual Reality Laboratory at the University of Michigan has been involved in research and development regarding the application of VR in industrial problems and other areas since 1993. Besides the use of HMD, BOOM, and CAVE systems for immersive applications, non-immersive VR and Web-based VR via VRML has become an area of high interest. The laboratory has explored the usefulness of VRML in design and manufacturing through a variety of studies and projects. The creation of a virtual model is a labor intensive and time consuming task. Even if a three-dimensional model already exists (e.g., as a CAD model), additional efforts are required to derive a polygonal representation with acceptable polygon count and to define supplementary information regarding appearance, lighting, functionality, and other properties. The fast generation of a virtual model, also known as rapid virtual prototyping, is still a topic of ongoing research and development.

Shen et al in **2001** [35] used Augmented Reality (AR) based urban planning as against the traditional graphics based virtual reality. The problems in the process of algorithm application, such as location of virtual objects were resolved through AR. An example is to fuse the virtual buildings of different sizes or sites into a real 3D scene. The system hardware included a digital camera and an Alpha-500 workstation. The software for the application was written in VC++.

Ling et al (**2002**) [36] worked on interactive finite element analysis using virtual reality. Using VR in interactive FEA, gives the pre-processor and the post-processor a good method to describe the model and results in a 3D world, which leads to a more intuitive and accurate understanding of the design and the construction. Applying VR in FEA and executing interactive FEA for the design and construction present a comprehensive and intuitive method for the understanding of complex problems.

Setareh et al in **2005** [37] developed a virtual structural analysis program by linking a visualization based routine using the VE library and structural analysis software Sap4. The interfaces were immersive in nature and were found highly satisfactory. The cycle of creating a model, simulating and animating can be repeated to test the changes to the structure. The main objective was to develop a system that enabled the user to interact with building structures in a totally immersive virtual environment.

Also in 2005, **Setareh** et al [38], reviewed a virtual reality based structural engineering software prepared at Virginia Tech. The software can be effectively used as an experiential teaching/learning tool. The portable immersive VR system was used for this purpose. Using the head-mounted-display (HMD) one student was immersed in the virtual environment while others were observing the same scene projected on the screen. Different topics related to the seismic behavior of building structures were considered. Case studies of Buildings with Re-entrant Corners, Soft Story etc. were done by creating a and subjecting it to the 1994 Northridge Earthquake. The torsion of the building due to its geometry was noticed. Students were asked about how the problem could be corrected and various corrective measures were tried. For each case, the student with the HMD moved around the building to better observe the effects of his/her changes on the structural behavior.

Jankovic et al [39] tested emergent models for structural analysis using virtual reality. In general, the complexity of the system such as truss, does not allow the programmer to easily model all of its states. One of the aims of the project was to investigate the possibility of using interaction of simple component models to create the model of the complex system, which emerges by itself, without directly programming it. This type of modelling is referred to as emergent, or bottom-up modelling. Another aim of the project was to abandon conventional solution methods used in the theory of structures, and to develop analogue models of structures in virtual reality from which the forces can be measured, rather than calculated.

2.5 STRUCTURAL ASPECTS OF WIND STORMS

Wind storms typically called cyclones are a major natural hazard and need to be addressed as a part of disaster mitigation efforts. Structural intervention is required to prevent failure or to minimize the damage due to excessive forces compared to the capacity of the members. Extensive research has been carried out across the world with robust codes and standards in place.

ASCE Task Committee on wind tunnel studies (**1996**) [40] stated that many codes of practice allow use of wind tunnel model testing for developing wind load information to supplement or replace code wind loads. Thus it becomes necessary for the engineering community to know when wind tunnel model tests are necessary and to become familiar with the underlying principles and procedures to ensure meaningful information. Candidates for wind tunnel tests are buildings which have an unusual sensitivity to wind particularly when a significant part of the wind induced response is dynamic. Examples are tall, slender and flexible buildings, or buildings with unusual aerodynamic shape, and buildings on unusual terrain and topographic features. The guidelines give the type of test for a particular type of information that is sought.

Picardo and Solari (2000) [41] proposed a single formulation of expression for along-wind, cross-wind and torsional responses to a generalized gust factor.It presented the wind-loading model and used a generalized equivalent spectrum technique to derive closed-form solutions of the 3D wind-excited response of slender structures and structural elements. The relative simplicity, the wide field of applications, the precision, and the reliability of this method make it suitable for rapid engineering calculations. The definition of equivalent static crosswind forces and torsional moments, analogous to the classic equivalent static along-wind forces, is a relevant tool for introducing these formulas in building code standards.

Zhou, Kijewski and Kareem compared various international codes in June **2002** [42]. Most international codes and standards utilize the "gust loading factor" (GLF) approach for assessing the dynamic along-wind loads and their effects on tall structures. Although a similar theoretical basis is utilized in these formulations, considerable scatter in the predictions of codes and standards has been reported. Kijewski and Kareem (1998) compared different international codes and standards with experimental data obtained in a wind tunnel using a high-frequency base balance. Their comparison also included the across-wind and torsional responses. In comparison to ASCE 7 and NBC, AS1170.2 provides the lowest estimates of the GLF, base bending moment, and the acceleration since it prescribes lower turbulence intensity, gust energy factor, and mean wind load. However, NBC employs the highest gust energy factor, which yields the largest value of GLF, base bending moment, and acceleration.

Hanzlik and Diniz (2005) [43] stated that for a building with specified orientation, the wind direction for which the response to wind is most unfavorable does not necessarily coincide with the wind direction for which the wind speeds are strongest. For most ordinary buildings, the ASCE 7 Standard specifies for the directionality factor a blanket value of 0.85 for all buildings and building components, even though the actual value depends upon the extreme wind climate, It also accounts for uncertainties in the factors that define wind effects e.g., wind speeds, peak wind effect coefficients, terrain roughness, quality of wind tunnel simulation, building orientation. The methodology yields estimates of wind effects that are more realistic than those based on the conventional building code approach for calculating wind effects.

Also in 2005 **Chen and Kareem** [44] showed that for buildings with one-dimensional mode shape in each primary direction, building response in each respective direction can be studied separately without the consideration of cross correlation of wind loads acting in different directions. However, buildings with complex geometric shapes or with structural systems having noncoincident centers of mass and resistance, or both, may result in 3D coupled mode shapes and experience coupled responses when exposed to wind loads. Furthermore, frequencies in the fundamental modes of vibration in three primary directions may be closely spaced. These situations warrant a 3D coupled response analysis framework for response predictions that take into account the cross correlation of wind loads acting in different directions and the intermodal coupling of modal responses.

Correa, Kareem et al (2006) [45] established a study to allow the first systematic validation of tall building performance in the United States using full-scale data in comparison with wind tunnel and finite-element models generally used in design. For each of the three tall buildings currently monitored in the city of Chicago, instrumentation is overviewed and wind tunnel and analytical modeling approaches are summarized. A comparison of the fullscale response features with design predictions was provided for the April 28–29, 2004 wind event. This comparison indicated that with respect to fundamental periods of vibration, standard modeling assumptions can reliably predict in situ periods of the uncoupled steel building. However, the assumptions made in the modeling of the reinforced concrete building in the study cannot be wholly validated, possibly due to levels of cracking assumed in the various service states not yet being realized in full scale as well as with the possibility that the in situ modulus of elasticity is higher than that assumed in the model

2.6 MITIGATION ASPECTS FOR WIND STORMS

Loss of property across the globe has been about 38% more due to wind storms than that due to earthquakes. Hence the mitigation aspects for reducing the effects on buildings in hurricane prone zones are critically important. International programs and research have been more focused on earthquakes hazards. Research on structural aspects to counter wind storms is complex as full scale tests are costly and impossible. Besides the collection of wind data too is difficult to maintain and monitor. Some of the critical issues have been presented in the work mentioned herein.

Unanwa and McDonald (2006) [46] : mentioned that the wind damage resistivities of buildings impacted by hurricanes and tornadoes depend primarily upon the strength of the connections between the building components, the nature of the components, the architectural design features, and the nature of the immediate environment. Using previously derived building wind vulnerability relationships based on upper and lower damage probabilities of building components and connections, called damage bands, and building relative wind resistivity indices based on expert experience, this paper presents detailed procedures for predicting wind damage to individual buildings, portfolio analysis, and wind damage mitigation. The maximum change in damage degree was found to occur at a windspeed of about 51.4 m/s (115 mph), The most significant wind performance parameters are summarized as follows: roof covering, roof sheathing, roof span, roof structure, exterior wall system, exterior door, percent wall occupied by exterior doors and windows, window glass type, presence of skylights and shutters on exterior doors and windows, and absence of overhead doors, and nearby debris sources such as gravel or roof-mounted equipment on nearby buildings.

Coulbourne et al (2002) [47] discussed that the design of shelter structures has received little attention from the engineering community since the days of nuclear fallout shelters, until the development of guidance for community shelters for cases of extreme wind events was released by FEMA in July 2000 (FEMA 361).

- Continuous structural load path from the building envelope to the structural frame and foundation,
- Wind pressure resistance of components and cladding
- Debris resistance of components and cladding.

The practice of designing structures to resist the effects of extreme winds consists of several steps:

- Determining the design wind event,
- Designing for wind pressures on the main wind force resisting system,
- Designing for wind pressures on the components and cladding,
- Designing the connections between the building elements, and
- Designing the building envelope to resist missile/debris impact caused by the design wind.

Wu, et al (2006) [48] defined robustness criteria for control systems in high rise buildings. The newly constructed Taipei 101 building in Taiwan with the total height of 509 m is one of the examples. Under wind excitation, the flexibility induced from their slenderness might cause excessive displacements and accelerations on which building serviceability and human comfort depend. To effectively reduce such excessive responses, it has been well recognized from much literature that the use of structural active control serves as a good alternative. Two typical issues concerning the control design process toward mature application shall be addressed. The first issue is regarding the accuracy of system prediction. In actual implementation of active control, one of the critical factors in system prediction, is mainly due to the integrated effect of active device (actuator) with structure, the so-called control-structure interaction (CSI). The second issue to be addressed is the profound consideration of robustness for controllers. A control design process incorporating robustness criteria was developed specifically for wind-excited high-rise buildings and its feasibility was experimentally verified by implementing on a four degree-of-freedom scaled building model.

Yang et al (2004) [49] presented an overview for the response control of wind-excited tall buildings. The building considered is a 76-story 306 m concrete office tower proposed for the city of Melbourne, Australia. The building is slender with a height to width ratio of 7.3; hence, it is wind sensitive. Either active, semiactive, or passive control systems can be installed in the building to reduce the wind response. In the case of active control systems, either an active tuned mass damper or an active mass driver can be installed on the top floor. In the case of passive or semiactive systems, such as viscous dampers, viscoelastic dampers, electrorheological, or magnetorheological dampers, etc., control devices can be installed in selected story units. Control constraints and evaluation criteria are presented for the design problem.

2.7 FIRE LOADS ON STRUCTURES

Prescriptive methods using Fire loads based on tests carried out in the 70s are being questioned and performance based methods are being adopted world wide. The construction materials as well as the technology has changed to the extent that the prescribed fire loads are highly conservative. Research has thus been focused on performance based objectives.

Chow W.K in **1998** [50] has conducted a numerical study on a large fire in a high rise building in Hong Kong. The fire and the building elements along with zone-model are simulated. The fire load density had exceeded the Fire Safety Department's (FSD) limit of 1135MJm^{-2} as it had burned for > 20h. Zone-model simulation in CFAST indicated that the lift shaft under construction had acted as a smoke vent and spread the fire to other levels of the building. The smoke temperature at the fire office increased to 280°C and at the lobby to 150°C. Two types of fire models are used to simulate the fire: the zone model CFAST and Computataional Fluid Dynamics (CFD) model PHOENICS. There are limitations in using both. In the CFAST model there is a limitation in simulating the smoke spreading in the high lift shaft. Deviations in the predicted smoke filling time are expected. There are no such limitations for the plume reaching top of the lift shaft using the CFD field. model.There are no such limitations on the time delay for the plume to rise to the top of lift shaft while using CFD field model. From the results, it can be seen that a lift lobby is important as it would prevent the fire from spreading. The CFD models give the distribution of temperature 118C-412°C while the zone models give the average value 227°C and 249°C. The average of both are fairly close.

Milke J in 1999 Sunil kumar et al in 1997 [51] carried out a fire load survey at Kanpur on 8 office buildings. The moveable contents contributed to 88% of total fire load. The study has concluded that the average fire load for all buildings was 348MJ/m^2 and standard deviation was 262 MJ/m^2. This paper provides an overview of the state-of-the-art engineering methods for structural fire protection applications that could be incorporated into an engineering standard. Engineering analysis of the response of fire-exposed structural assemblies involves consideration of: 1) Fire exposure conditions 2) Material properties at elevated temperatures; 3) Thermal response of the structure; 4) Structural response of the heated assembly. The relationship of these four issues in a performance-based design approach for evaluating fire resistance is examined. The fire endurances of the undamaged siliceous aggregate concrete column and girder are estimated to be greater than, respectively, 90 and 120 min under the ISO 834 fire exposure. However, once portions of the concrete cover for the reinforcement have been stripped off, the structural capacities of the damaged columns and girders can quickly be compromised, The fire endurance times for the column and beam are determined as the times for the main reinforcing steel to reach critical temperature @932°F (500°C), For the damaged column, the fire endurance time ranges from 25 min to 50 min. For the damaged girder, the fire endurance time ranges from 12 min to 20 min, (based on the upper boundASTM E-1529 and lowerbound fire ASTM E-119).

Huang, Burgess et al in 2000 [52] described a three-dimensional, nonlinear finite-element procedure for modeling composite and steel-framed building behavior in fire. In this approach, composite steel-framed buildings were modeled as an assembly of finite beam-column, spring, and slab elements. The beam-columns were represented by two-noded line elements. The cross section of the element was divided into a number of segments to allow consideration of temperature, stress, and strain through the cross section. A two-noded spring element of zero length was used to model the characteristics of steel member connections. The slabs were modeled by using a layered flat shell element based on Mindlin/Reissner theory, in which each layer can have a different temperature and material properties. Predictions from the model were compared with experimental results, both from isolated element tests and from a major full-scale fire test performed in the experimental composite building at Cardington. The model was capable of predicting the response of composite steel-framed buildings in fire with reasonable accuracy. The procedure was found to have good computational stability, which is very important in the context of problems incorporating very large deformations.

Wang and Kodur in 2000 [53] stated that when exposed to fire, steel loses stiffness and strength; to limit this loss of strength and stiffness, external fire protection is provided to the steel structural members to satisfy required fire resistance ratings. However, the current practice of providing fire protection is based on the behavior of single elements under idealistic situations; as such, it is conservative and does not represent the realistic fire behavior of real structures. These studies mainly focused on understanding the realistic fire behavior of complete structures and on developing innovative systems with inherent fire resistance. Feasible solutions are now emerging, in which the external fire protection may be completely removed, in certain situations, without compromising the fire safety of steel structures. This system uses the conventional composite slab/steel beam flooring system in conjunction with concrete-filled steel tubular columns. Research studies at the United Kingdom's Building Research Establishment and the National Research Council of Canada suggest that it might be possible, through proper design, to eliminate fire protection for steel in this system.

Wong in 2001 [54] presented the use of plastic analysis methods for the calculation of critical temperature distribution for structures at collapse under fire conditions. The demand for a rational, performance-based approach to design of structures under fire conditions is increasing as more design codes across the world have progressively abandoned the conservative and prescriptive approach for fire design. The simple concept of plastic load factor for collapse analysis is extended to elevated temperature analysis which leads to the development of the unit load factor method for critical temperature factor calculation. The critical temperature factor is used to indicate the factor of safety for a structure under certain fire scenarios. In this approach, linearly varying distributed loads can be included in the analysis. The upper-bound approach is based on the conventional collapse mechanism calculation method, modified to include the temperature-dependent material properties.

Xudong and Teng-Hooi in 2004 [55] tested six specimens with different concrete cover (CC) thickness (10–30 mm) to investigate the influence of the CC on the properties of reinforced concrete flexural members exposed to fire. The specimens were heated on their bottom and two lateral surfaces. From these test results, it is shown that the bottom CC has significant influence on the specimen ultimate loading capacity, but the extent of this influence will decrease with an increase in the CC thickness. Thus, it is improper to excessively increase the bottom CC thickness to improve the specimen fire resistance. The lateral CC has a less beneficial effect on the specimen fire resistance compared to the bottom CC. A concept of the equivalent CC thickness is proposed in this paper to better predict the effect of the CC on the flexural capacity of reinforced concrete members exposed to fire. With continuous widening of the concrete cracks at the tensile zone, the effect of the bottom CC thickness will decrease gradually. This is particularly so when the bottom CC thickness is greater. Therefore, for important flexural members or those with larger spans and sustaining greater vertical load, it is not practical to strengthen their fire resistance only through increasing the CC thickness

Lim and Buchanan (2004) [56] described the computer modeling of axially restrained, one-way reinforced concrete slabs in fire conditions. The study was carried out for slabs with pin supports and slabs with rotationally restrained supports. A single span, 5 m slab, carrying a uniformly distributed load was analyzed using a special purpose, nonlinear finite element program, *SAFIR*. The study for pin-supported slabs considered the effects of the position of the thrust force at the supports and different axial restraint stiffnesses. The effects of axial restraint on slabs with rotationally restrained supports were also investigated. The analyses found that the behavior of pin-supported slabs is very sensitive to the position of the thrust force at the end supports and the axial restraint stiffness. Slabs with rotationally restrained supports are predicted to have much better fire resistance than equivalent pin-supported slabs due to moment redistribution. The behavior of rotationally restrained slabs in fire conditions is less sensitive to the different axial restraint stiffness than pin supported slabs.

Johann et al (2006), [57] This paper summarizes recent research and provides a possible approach to integrating fire safety engineering into the design process for structural framing systems. Performance-based design of fire-safe structures is defined in terms of five activities, and a series of flowcharts organizes these activities in terms of sublevel functions and their interrelationships. The flowcharts also help to identify informational needs critical to performance based structural fire safety and to develop an understanding of the role of the fire protection engineer. the International Code Council ,ICC 2001 and the National Fire Protection Association NFPA 2002 have introduced performance-based provisions for fire safety as an alternative to existing model prescriptive codes. Moreover, Australia, the United Kingdom, New Zealand, Sweden and the Eurocode have already adopted performance-based approaches to structural fire safety. Structural engineers generally do not have the knowledge and experience necessary to analyze and make judgments about structural performance at elevated temperatures. Moreover, they lack the knowledge and experience to deal with uncertain fire conditions. Instead of mechanics-based study of performance under various fire and loading conditions, the building code system, with its prescriptive requirements and their enforcement, has assumed responsibility for structural fire safety. In fact, fire protection engineers are seldom members of building design teams; their participation is limited to exceptional circumstances or unique structures.

Following categories of structural modifications are considered, as summarized:

1. Passive protection of structural members by adding insulation, coatings, barriers, etc.;
2. Differences between the originally specified structural configuration and/or protection and the as-built condition;
3. Changes to the structural configuration and/or protection caused by prefire events such as earthquakes, blasts, accidental loss of protective material,
4. Changes to the structural configuration and/or protection subsequent to a fire.
5. Changes to the structural configuration and protection subsequent to a fire.

Fire performance criteria for a structural system may include:

- Limitations on member deformation or requirements for Serviceability.
- Requirements for load-carrying capacity ,prevention of collapse;
- Time to failure requirements: allow occupant egress and suppression activities ;
- Fire containment requirements:limitations on the impact of a fire on structural members distant from the fire and prevention of room-to-room fire spread.

The state of the fire is dependent upon the following parameters:

- Rate of fire growth;
- Total rate of heat release into the compartment;
- Modes of heat transfer to compartment contents (convection radiation);
- Burning regime preflashover or postflashover, ventilation controlled burning
- Ventilation and opening factor parameters.

Nestor in 2007 [58] stated that concrete and masonry are noncombustible materials with relatively low thermal conductivity. Their structural design often results in comparatively heavier and more massive members that provide a desirable heat sink for fire resistance. Because of this thermal mass effect, concrete and masonry construction has also been routinely used for thermal insulation and/or fire barriers. Similar to other construction materials, concrete and masonry experience property degradation with increasing temperatures, as well as visible cracking or spalling damage in concrete. The densities of normal and lightweight concrete effectively remain unchanged up to a temperature of about 800°C, when normal weight density begins to rapidly deteriorate by approximately 25–50%. At room temperature, the baseline concrete compressive strength, fc can vary between about 20–55 MPa. Similar to steel, the strength reductions of these normal strength concretes are negligible up to about 300°C material temperature. The concrete strength losses greatly increase at higher temperatures. In the range of 500–600°C, the stiffness (modulus of elasticity) and strength reductions due to heating can be roughly about 50%. The standard ASTM E 119 fire exposure and its acceptance criteria for undamaged new construction are generally applied. These simple calculation methods and design aids were developed for single members or isolated subassemblies in a fire compartment subjected to a standard E 119 fire with idealized boundary conditions i.e., without overall structural system interactions and are not intended to be a predictor of structural failure in buildings for actual loads or fires The identification of specific ultimate structural failure modes or their potential progression to collapse mechanisms cannot be obtained from these formulations. However, these methods enable an efficient and generally conservative way to expand the fire resistance ratings for members and assemblies that do not directly match published tests. In this manner, the prescriptive code requirements can be conveniently and safely met. The assumptions, limitations and constraints of the prescriptive fire resistance methods based on E 119 ratings can be superseded by a more advanced performance-based fire engineering design intended to achieve a more accurate solution to overall structural fire safety.

2.8 DAMAGE AND REPAIR FOR FIRE

Damage of structural elements may or may not be repairable depending on the location and extent. Repairing techniques are quite advanced with many new materials and design methods which would regain the strength almost as close to the original level if not more. Construction and structural engineers are gaining ground in an area considered once exclusively of fire protection engineers. Collaborative efforts of all three have lead to efficient techniques.

Haddad et al(2007) [59] proposed several repair techniques for restoring the structural capacity of heat-damaged high strength reinforced concrete shallow beams using advanced composites. A series of 16 under-reinforced concrete hidden beams were cast, heated at 600°C for 3 h, repaired, and then tested under four point-loading. Tests were conducted to study the effectiveness of externally applied composite materials on increasing the flexural capacity of beams. The composites used include high strength fiber reinforced concrete jackets; ferrocement laminates; and high-strength fiber glass sheets. The beams repaired with steel and high performance polypropylene fiber reinforced concrete jackets regained up to 108 and 99% of the control beams' ultimate load capacity, with a corresponding increase in stiffness of up to 104 and 98%, respectively. The beams repaired with fiber glass sheets and ferrocement meshes regained up to 126 and 99% of the control beams' ultimate load capacity, with a corresponding increase in stiffness of up to 160 and 156%, respectively. Most of the beams repaired showed a typical flexural failure with very fine and well distributed hairline cracks in the constant moment region.

Chow (2002) [60] proposed a Fire Safety Ranking System for assessing the fire safety provisions in existing high-rise nonresidential buildings in Hong Kong. The objective is to investigate how far the fire safety provisions in those existing buildings deviate from the expectation of new codes. Suitable fire safety management can then be worked out in the transition period, based on the scores. Local fire codes were reviewed first to decide what should be the attributes and their weightings. From the reviewing results, three groups of attributes were proposed. These are the passive building construction, active fire protection systems (fire services installation) and key risk parameters, all following the local fire safety requirements.

Ufuk in 2007 [61] summarized an engineering evaluation of the extent of fire damage to a concrete structure under construction. The fire occurred in a portion of the reinforced concrete structure and visibly damaged a load bearing exterior foundation wall. The purpose of the assessment was to promptly evaluate the in situ condition of the wall and recommend necessary repair or replacement options prior to commencement of backfilling and the concrete construction to be supported by the subject wall.

Xin et al in 2007 [62] developed an assessment method for fire-damaged high-strength concrete (HSC) structures that is convenient for postfire repair. The effects of concrete strength and aggregate type on the postfire residual compressive strength of HSC were studied. Subsequently, the changes in durability and microstructures were investigated by the rapid chloride-ion penetrability test, the mercury intrusion porosimetry test, and scanning electron microscopy observation.

Elghazouli and Izzuddin in 2004 [63] studied the behavior of lightly reinforced concrete members under fire conditions, focusing on the failure state associated with rupture of the reinforcement. The work transpires from the need to examine the underlying mechanisms related to the failure of composite floor slabs, which become effectively lightly reinforced in a fire situation due to the early loss of the steel deck.

Tan and Yae in 2003 [64] developed a method to predict fire resistance of RC columns. Until now, the determination of fire resistance of reinforced concrete columns has essentially been based on tabulated data. Both uniaxial and biaxial bending of columns is considered. The computer code *SAFIR*, developed at the University of Liege, was used to analyze reported experimental results and to simulate the deformation response.

Sakumoto in 1999 [65] presented new fire-protection materials. These were an intumescent coating that foams when heated and a fire protection ceiling provided with plaster boards to isolate heating coming from the room inside, allowing elimination of the need for fire protection on the steel frames above the ceiling. The temperatures of steel frames subjected to heating by fire were then calculated. Steel materials studied for comparison in the research on the fire resistance of steel frames were fire-resistant steel, for which the high-temperature strength of the steel itself is raised; structural stainless steel, which has excellent heat resistance; and conventional steel The failure temperature was as follows in the case of the span-direction frame: Conventional steel: 620°C; Fire-resistant steel: 740°C; Stainless steel: 790°C.

2.9 STRUCTURAL ASPECTS OF FLOOD CONTROL

Most major rivers have been the cause of devastation of life and property which exceeds that caused by earthquakes. Flood risk management is a multifaceted activity involving all aspects of civil engineering. Studies and reports from real-life experiences and inferences help mitigate the disastrous effects of floods.

Al-Weshah and El-Khoury in 1999 [66] developed flood mitigation models for Petra in the southwest region of Jordan, 200 km south of Amman, between the Dead Sea and the Gulf of Aqaba. Petra was carved in sandstone canyons by the Nabatean over 2,000 years ago.Today the city is a major tourist attraction, its monuments being considered the jewels of Jordan. Using the model, flood flows and volumes are estimated for storm events of various return periods. To alleviate the impact of floods on tourism in Petra, several flood mitigation measures are proposed. The impact of these measures on flood peakflow and volume is evaluated. These include afforestation, terracing, construction of check and storage dams, and various combinations of these measures. The flood simulation model predicts that the measures can reduce flood peak- flows and volumes by up to 70%.

Jaffe and Sanders (2001) [67] examined the inland flooding for excessive precipitation and surface runoff remains the cause of extensive damage, loss of property, and human suffering worldwide. In this project, an engineered levee breach was considered as a measure to mitigate floods, and design attributes of the optimal levee breach were examined. The engineered levee breach creates depression waves that destructively interfere with the flood wave to crop peak flood stages. Furthermore, the engineered levee breach permits control over flood stage at a point along a channel where flood stage reduction is most beneficial in terms of hazard mitigation. The roles of flood plain storage, breach size, flood discharge, flood duration, and breach timing in the optimal design of engineered levee breaches were examined using a shallow-water model, and a strategy to design levee breaches is obtained by scaling the model results. It is shown that substantial flood stage reduction can be achieved with an engineered breach. This approach to flood fighting could prove useful in the mitigation of relatively short, extreme floods.

Scawthorn C, Flores P et al, (2006), Part I & II [68] stated in a two-part paper an overview of the HAZUS-MH Flood Model and a discussion of its capabilities for characterizing riverine and coastal flooding hazard. Effective flood management requires a coordinated, integrated approach that uses structural defenses at the edge of the floodplain as well as wise land use planning and restraints on construction within floodplains. The HAZUS Flood Model is based on an integrated set of flood hazard analysis algorithms and allows estimates of flood losses also. Part II reports on the damage and loss estimation capability of the Flood Model, which includes a library of more than 900 damage curves for use in estimating damage to various types of buildings and infrastructure. Based on estimated property damage, the model estimates shelter needs and direct and indirect economic losses arising from floods. Analysis for the effects of flood warning, the benefits of levees, structural elevation, and flood mapping restudies are also facilitated with the Flood Model.

Becker , Johnston et al (2007) [69] studied the effects of flood due to the failure of a local dam. On October 6, 1999, a large rock avalanche from Mount Adams on the west coast of the South Island, New Zealand, fell into the Poerua Valley. The landslide blocked the river valley, damming the Poerua River, and creating a large lake. The potential for overtopping and failure of the landslide dam presented a potential dam-break flood hazard that was assessed as posing a serious danger to Poerua Valley downstream. The dam eventually failed 6 days after it was formed. Fortunately, the resulting flood was largely confined to the river channel and flood-plain areas, causing little damage and no deaths.

2.10 STRUCTURAL ASPECTS OF BLASTS LOADS

Terrorist attacks worldwide have made the engineering community rise up to this new threat to the structures and the devastation that they cause. Important buildings which are identified as potential targets are now being designed on the performance based philosophy. Preventive measures such as blast resistant curtain walls are being inbuilt at the the new construction stage itself. Progressive damage is being examined thoroughly so that failure of critical members does not trigger collapse of the structure beyond repair.

Corley, Sozen et al (1998) [70], gave recommendations for multihazard mitigation based on the Oklahoma city bombings triggered by a truck bomb in Murrah building. It is shown that progressive collapse extended the damage beyond that caused directly by the blast. Based on FEMA, structural systems that would provide significant increase in toughness under catastrophic loads such as earthquakes and blasts, are identified. One such system is compartmentalized construction, in which a large percentage of the building has structural walls that are reinforced to provide structural integrity in case building is damaged. Another type used in high seismicity areas is the special moment frame system or the dual system with special moment frames. These systems provide the mass and toughness necessary to reduce the effects of extreme overloads.

Naito and Wheatonin 2006 [71], presented a methodology for assessing the performance of structural elements subject to explosive loading. The method combines basic section analyses, equivalent single degree of freedom (SDOF) modeling, and a static finite element pushover analysis to calculate the blast resistance of an existing shear wall subject to an external explosion. A static pushover analysis is conducted to identify regions of vulnerability. The second floor wall is identified as the location of failure and is modeled as a system, which includes the stiffness contributions of adjoining wall sections, and also as a component with fixed ends. Pressure–impulse curves are developed to quantify the blast resistance of the wall relative to various levels of damage. The component model is found to underpredict the blast resistance by 7% when compared to the system model for impulsive demands. Simplified energy methods are also shown to bound results of the SDOF resistance curves

Weggel, Zapata et al in 2007 [72] stated that recent terrorist events have underscored the importance of designing glass curtain walls as engineered systems. For most building owners, however, it is usually an unjustifiable expense to have a curtain wall designed to provide a high level of blast resistance. This paper investigates the serviceability of an economical, nearly conventional, glass curtain wall system that provides a low level of blast resistance in appropriate applications. The calibrated model is used to perform modal and transient analyses that are compared to experimental free vibration responses. The model was found to provide results that agree well with experimental responses, indicating that the primary properties have been adequately defined for reliable serviceability investigations.

2.11 ASPECTS OF TSUNAMI

With the recent activity of plates across the globe, the danger of Tsunami manifested in 2004 on the coasts of most south east Asian countries causing devastation of an unexpected magnitude. Governments are now rising up to the threat and working out guidelines with the help of federal agencies.

International handbook of earthquake and engineering seismology [73] gives detailed explanation of causes of tsunami, due to typical interplate earthquake and tsunami earthquakes. Size of tsunamis has been explained which includes tsunami magnitude and intensity. Various types of tsunami magnitude scales have been defined such as Imamura-Iida scale, a formula has been derived by Iida for approximately evaluating tsunami magnitude and also tsunami intensity was defined by solo views (1970). Instrumental measurements for tsunamis have been explained, which includes the measurements of tsunamis with the help of tide gauges operated by national ocean services since 1850's.

FEMA Coastal construction manual [74] summarizes the selection of mitigation strategy for the site planning which includes different mitigating ways such as avoidance of inundation areas, slow water currents, steer water forces and block water surfaces. Effective site planning in coastal areas according to the vulnerability to tsunami hazards has also been mentioned. An ASCE team of engineers visited the west coast of Thailand to investigate damage from 26th December 2004 Tsunami and to document the effects on building structures.

An effort to draft a tsunami code in India has been included, due to the increasing frequency of earthquakes in India. **Guidelines for designing tsunami resistant structures** have been given **by the Tamil Nadu Government in 2005** [75]. Detailed site selection, planning and anchorage guidelines have been given.