CHAPTER 2

REVIEW OF LITERATURE

2.1. General:

Structural Safety involves, in general, uncertainties in loading, structural strength and structural response. A brief review of literature and a comprehensive bibliography was published in 1972 by the Task Committee on Structural Safety of the American Society of Civil Engineers.

The literature on structural safety is briefly reviewed with particular reference to (i) Load Analysis, (ii) Strength Analysis, (iii) Dynamic Structural Analysis, (iv) Structural Design and Reliability Analysis, (v) Ergonomics and (vi) Structural Codes.

2.2. Load Analysis:

The following loads are generally considered in the analysis of structures. (i) Dead Loads and Imposed Loads, (ii) Wind Loads and (iii) Seismic Loads. The Fire Loads are also assessed for the safety of structures. The dynamic effects of certain live loads which produce vibrations by impact such as moving trains, trucks, cranes and blast loads etc. are to be considered wherever necessary.

2.2.1. Dead Loads and Imposed Loads: The dead load in a building comprises the weight of all walls, partitions, floors and roofs and includes the weights of all other permanent constructions in the buildings. The imposed loads cover the loads assumed to be produced by the intended use or occupancy of a building, including
the weights of movable partitions, distributed, concentrated loads, loads due to impact and vibration and dust loads.

The first Interim Report of the Steel Structures Research Committee, London\textsuperscript{130} by C.M. White on "Survey of Live Loads in Offices" described the relationship between areas of different sizes and intensities of loadings and discussed the effects of different tenancies on average loading. Dunham (1946)\textsuperscript{107} has analysed the surveys of live loads in two office buildings in Washington, D.C. in 1945 conducted by U.S. Public Buildings Administration and developed and proposed a system for the reduction of basic live loads with increased area. Apcear (1948)\textsuperscript{21} has appraised the problem of loadings in factories.

Horne (1951)\textsuperscript{169} gave a simple load model to predict design loads for areas of various sizes and reviewed load reduction versus area characteristics, which was adopted by Canadian and Mexican Codes. Dunham et al (1952)\textsuperscript{108} reported sampling surveys of departmental stores and factories in New York and Washington. Freudenthal (1956)\textsuperscript{136} has distinguished between simple loads (varying in magnitude only) and complex loads (varying in magnitude, location and distribution). He recommended statistical approach to design loads except for the cases such as warehouse floor loads, storage tank pressures and maximum train loads on rail road bridges where the maximum load is governed by arbitrarily established limits and occurs relatively frequently. Based on previous surveys, he suggested that highway bridge live loads can be represented by a lognormal distribution and aircraft freight loads by a Poisson distribution. The International

Jeuffred (1960) surveyed live loads in office buildings located in Mexico. He fitted extreme type I distribution to the weights of personnel and normal distribution to weights of furniture, but measures of goodness of fit are not given. Mitchell and Wood Gate (1966-67) surveyed for ERS, England, floor loadings in office buildings, retail premises and domestic buildings. They have given load concentration factors to be multiplied by loading intensities to obtain equivalent uniformly distributed loads. They have concluded that the frequency distributions of loading are not normal and no distribution consistently applicable to all bay sizes could be fitted.

Hasofer (1966) analysed the survey of actual live loads in buildings carried out at University of Melbourne and suggested that live loads can be represented by a two-dimensional stochastic process with independent increments, the prototype of which is the compound poisson process. For National Bureau of Standards (USA), Bryson and Gross (1967) have conducted pilot survey of imposed floor loads and fire loads in 2 modern office buildings. The data were coded and recorded in a format consistent with an appropriate computer programme for automatic data processing. CIB (1967) has attempted to standardise
methods of assessment of industrial loadings. Karmann (1969) has
in Hungary has surveyed the loads in office buildings, concluded
that for most structural purposes a load may be considered
'permanent' when it exists for more than 5% of the operation
time of the building.

Pier (1971) has suggested a model for all practical pur-
poses, according to which the load on a given area remain con-
stant between changes of occupancy and those changes take place
as a simple poisson process. The model is specified by the mean
rate of change of occupancy (which is a function of the area)
and the distribution of the instantaneous load for any occupancy.
Ferry Borges and Castenheta (1971) have derived a method for
the joint distribution of loads that occur simultaneously at least
once during the life of the structure. Ferry Borges and
Castenheta (1972) have examined the vector load effects which
cannot be reduced to a scalar equivalent. Mitchell (1972) has
classified the loads into 9 groups based on type of occu-
pancy. He remarked that all the occupancies for which surveys
have so far been done show a decrease in the load with the
increase in the size of the floor zone considered. Greene (1972)
described data collection techniques for live load surveys
making extensive use of stochastic models and stratified
sampling techniques. Pier and Cornell (1972) have described
sustained load models and maximum sustained load models.

Johnson (1972) assessed the then practice of determining
the magnitudes and characteristics of dead, live and construction
loads, which a structure must support and pointed out some factors which appear to have been overlooked and made suggestions for new approaches and criteria. Apeland (1972) has compared number of reduction formulae viz. Michell and Woodgate, Danish Code, CIB recommendations, Norwegian Code, Dunham and Swedish Codes and pointed out wide scatter and stressed to establish a reliable criteria. ISO (1972) has attempted to standardise methods of assessment of industrial loadings. 

Pier and Cornell (1979) have developed probabilistic models of the temporal and spatial variability of sustained live (gravity) loads in office buildings. These models were fitted to the results of live load surveys conducted by Mitchell and Woodgate. The load intensity model can be used to predict temporal variations of structural effects caused by live loads. Poloheimo (1973) has analysed live load surveys of 20 office buildings conducted by Finnish Ministry of Domestic Affairs. The load intensities were derived. The frequency histogram of transient load (No. of persons) was given instead of the load. McGuire & Cornell (1974) have examined the effect of influence of surface shape, effect of the area supported, division of supported area into independent "tenants" or offices, and the effect of the number of supported floors. The Equivalent Uniformly Distributed Load (EUDL) which produce the same load effect fractile as actual spatially varying live loads was computed for several load effects (shear, moment and axial force) for various supported areas. The fractiles of the maximum total EUDL i.e. peak effective load that a structural
element may experience during its life were compared to the design loads in use in U.S. (1972) and Canada (1970). Several methods were proposed for incorporating those results into structural code formats to obtain design load effects with a consistent probability against being exceeded.

Sentler (1974) analysed the live load survey data of modern office buildings in Sweden. The offices were split into six functional groups according to the use and mean load intensities were evaluated for each group. He observed that there is a decreasing effect on the mean load as the floor area increased. Dayeh (1974) has conducted a pilot survey to establish and test the procedures and to develop analysis for comprehensive survey of live loads in office buildings in Sydney, Australia. He observed that furniture accounted about 80% of the total load and partitions accounted up to 45% of total load in small rooms but their contribution decreased in larger rooms and the load due to personnel was less than 10% of total load. Load intensities were assessed both as a function of room use and zone (internal/external) area. He reported a mean occupancy duration of 12.3 years for different tenancies.

Joint Committee on Structural Safety (1974) has developed load modelling for individual loads. As a part of international efforts to develop model design codes, the problem of load definition and classification has been studied in considerable detail by Mathieu (1975). Ellingwood & Culver (1977) studied various probabilistic models to
analyse the National Bureau of Standards survey results (1976)\textsuperscript{86} and developed design loads and compared the NUDL and load reduction factors with those obtained from U.K. (1973)\textsuperscript{214} data. They have concluded that the live load reduction factors specified by ANSI - A58.1 - 1972 reduces the design loads too rapidly as area increases and allows too high or maximum reduction.

Wen (1977)\textsuperscript{419} presented a load combination analysis considering load occurrence rate, intensity variation (random), duration of each occurrence and simultaneous occurrence of different loads and derived the probability distribution of maximum combined load effects over a given time interval. He obtained the distribution parameters as a function of first two moments and occurrence rates of individual load based on theory of statistics of extremes. Corotis and Doshi (1977)\textsuperscript{76} have analysed the major surveys by Dunham, Brekke and Thomson. Veneziano et al (1977)\textsuperscript{400} have presented a vector process approach for linear combinations of Gaussian processes.

Ranganathan and Dayaratnam (1977)\textsuperscript{332} have surveyed live loads in office rooms at IIT, Kanpur, and concluded that live loads follow lognormal distribution and reported a mean value of 120.7 kg/m\textsuperscript{2} with a c.o.v. of 0.386.

Kuureghian (1978)\textsuperscript{214} presented an approach for the evaluation of first two moments of stochastic loads and their combinations. Application of this method to theoretical load models, including stationery Gaussian, Poisson square wave and filtered Poisson processes were studied. Computed results for various combinations of these processes were shown to be in good agreement.
with Monte Carlo simulation. Gregorin and Turkstra (1978) have developed load modelling for combinations of loads. Wen (1979) has examined the accuracies of the available formulae relating the statistics (Mean and Standard Deviation) of instantaneous intensity of sustained live loads to those of life time maximum values and proposed a method based on a Gauss-Markov Model for the sustained live loads in multistorey buildings. He concluded that the mean life time maximum equivalent uniformly distributed load (EUDL) for columns of multi-storey buildings may be greater than that of single storey case.

Larrabee (1979) has explored the accuracy of using the mean up crossing rate as an approximation to the desired probability distribution of the maximum of the sum of the two load processes during some design life time. Corotis and Jaria (1979) have examined the nature of the floor live load process comprised in general of all non-permanent gravity loads, including furniture and equipment (books, paper, tables, mechanical equipment etc.), people and movable partitions. They have analysed the physical aspects of live load behaviour to develop a model which would form the basis for further surveys and Code specifications. Turkstra (1980) has investigated the linear combinations of a large class of loads routinely encountered in structural design and developed rules for determining design values, to assure that the probability of exceedance of a sum of loads is approximately equal to the probability of exceedance of any load acting separately. He has concluded that the Rackwitz-Fiessler (1977) algorithm provides an efficient approach to combination analysis.
except when two or more very infrequent loads are combined.

Wen (1980) developed a method for the evaluation of load coincidence whereby probability of combined loads and lifetime reliability of structure under such loadings can be obtained. Wen (1981) has studied the clustering of loads, with the motivation that many natural or man made hazards which cause serious load effects or structural failure may be due to common sources. For example, severe storms may produce extreme winds, waves, snow and temperature loads, strong motion earthquakes may cause dynamic force and indirect fire loadings and in a nuclear structure, loss-of-coolant-accidents (LOCA) loading because of pipe breaks. These loads may have different arrival times, intensities and durations but may be clustered around a common point in time such that there is a higher chance of simultaneous occurrence and hence graver consequences than if an independence assumption is made. Based on a multi-variate point process presented - an analytical model to take such occurrence dependence into consideration. He developed methods for analysis of mean rate coincidence (simultaneous occurrence) probability of combined loads and reliability of structures under such loads as an extension of load coincidence methods suggested earlier by the author. Wen (1981) has further investigated the stochastic dependence in load combinations, in the context of summations of two pulse processes in which occurrence time, intensity and duration are allowed to be correlated within each process and between processes. He obtained approximate analytical solutions based on a load coincidence method and investigated...
Carlo simulations. He observed that while within load positive correlations may generally have only moderate effect on the combined load probability, between load dependence may be dominant factors and significantly increase the probability of threshold level being exceeded by the combined load.

Ditlevsen (1981) in his state-of-the-art paper on probabilistic modelling of man-made load processes and their individual and combined effects, focused on the critical modelling aspects of man-made static type, macro-scale load processes for application in failure probability calculations of highly reliable structures as are relevant in civil engineering.

Shinozuka (1981) has reviewed the load combination methodologies pertaining to simultaneous load occurrences. The probabilities of simultaneous load occurrence are rigorously derived under the condition that the arrival of each load effect was governed by Poisson Law, while its duration distributes either as an exponential or Erlang distribution. He derived an expression for the upper bound of the limit state probabilities under a relatively simple limit state condition combining the probabilities of simultaneous occurrence with corresponding up-cross ratings. CEB Commission 1, "Reliability and Quality Assurance" (1982) has presented and discussed different existing solutions viz. probabilistic level II approach, semi-probabilistic approach and empirical approaches for the fundamental combination of various actions for ultimate limit states.

Krishnamurthy (1983) surveyed occupancy loadings in five
modern office buildings in Madras. The survey covered a total floor area of about 28,500 m$^2$ in 450 rooms to estimate the imposed and fire loads and concluded that (i) the variation of loads between the floors is not significant, but the variation within the floor for different rooms is significant due to the room usage, (ii) there is a marked difference between the EUDL which is based on structural effects in the supporting structure and the usual Room or Bay-load Intensities which are only nominal load intensities. The mean load concentration factor (ratio of mean Equivalent Uniformly Distributed Load (EUDL) to mean Bay Load Intensity (BLI) is as high as 2.0, (iii) the log normal distribution is found to be the most appropriate choice for describing the various loads and also fire severity, (iv) the observed 95% probable EUDL of 2.35 KN/m$^2$ in office buildings without separate store rooms is very much less than the present IS Code provision of 4.0 KN/m$^2$ for the design of such buildings which seems to be close to the 99.6% probable level.

Corotis & Tsay (1983) have derived a procedure for computing duration statistics as a function of load level i.e., the complete probability distribution of total time above a fixed reference load, duration of a single excursion up-crossing the reference load, and number of excursions for a fixed design lifetime. These quantities are necessary for computing long term deflection and settlement of structures critical load combinations for creep rupture materials such as wood.

2.2.1.1. Similarities, differences and analysis of different
load surveys: In most of load surveys imposed loads are expressed as average intensities in rooms or in bays or in selected floor zones or in influence areas or in whole building or structural stress resultants. Some surveys considered individual rooms as basic units.

Some investigators have studied the effect of room size on the load intensity and have come to a general conclusion that load intensity decreases with increase in room area. Recent studies reported mean, variance, frequency histograms of basic data. Some studies have tried to fit probability models. Some investigators have studied the combination of various loads.

Minimum design loads specified by various International Codes for various occupancies are shown in Table 2.1.

**TABLE 2.1 - Minimum Uniformly Distributed Imposed Loads (Kg/m²)**

<table>
<thead>
<tr>
<th>Occupancy of use</th>
<th>India</th>
<th>Great Britain</th>
<th>U.S.A.</th>
<th>West Germany</th>
<th>IS 1875 Code</th>
<th>Part II Ch. V</th>
<th>Draft Part I</th>
<th>1985</th>
</tr>
</thead>
<tbody>
<tr>
<td>Private rooms</td>
<td>200</td>
<td>204</td>
<td>105</td>
<td>200</td>
<td>150</td>
<td>300</td>
<td>200</td>
<td>150</td>
</tr>
<tr>
<td>Corridors</td>
<td>300</td>
<td>408</td>
<td>300</td>
<td>500</td>
<td>200</td>
<td>300</td>
<td>300</td>
<td>200</td>
</tr>
<tr>
<td>Guest rooms</td>
<td>200</td>
<td>204</td>
<td>105</td>
<td>200</td>
<td>150</td>
<td>300</td>
<td>300</td>
<td>200</td>
</tr>
<tr>
<td>Corridors</td>
<td>300</td>
<td>408</td>
<td>300</td>
<td>500</td>
<td>200</td>
<td>300</td>
<td>300</td>
<td>200</td>
</tr>
<tr>
<td>Class rooms</td>
<td>300</td>
<td>306</td>
<td>105</td>
<td>500</td>
<td>200</td>
<td>300</td>
<td>300</td>
<td>200</td>
</tr>
<tr>
<td>Corridors</td>
<td>400</td>
<td>408</td>
<td>300</td>
<td>500</td>
<td>200</td>
<td>300</td>
<td>300</td>
<td>200</td>
</tr>
</tbody>
</table>
### Table 2.1 (Contd.)

<table>
<thead>
<tr>
<th>Occupancy of use</th>
<th>India</th>
<th>Great Britain</th>
<th>U.S.A.</th>
<th>West Germany</th>
<th>IS0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Office buildings</td>
<td>200</td>
<td>200</td>
<td>200</td>
<td>200</td>
<td>200</td>
</tr>
<tr>
<td>Hospitals: Private rooms</td>
<td>250</td>
<td>250</td>
<td>244</td>
<td>200</td>
<td>200/300</td>
</tr>
<tr>
<td>Assembly Halls:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fixed seats</td>
<td>400</td>
<td>408</td>
<td>293</td>
<td>500</td>
<td>400</td>
</tr>
<tr>
<td>Movable seats</td>
<td>500</td>
<td>510</td>
<td>488</td>
<td>500</td>
<td>500</td>
</tr>
<tr>
<td>Stairs: Private buildings</td>
<td>300</td>
<td>306</td>
<td>488</td>
<td>350</td>
<td>300</td>
</tr>
<tr>
<td>Public buildings</td>
<td>500</td>
<td>510</td>
<td>488</td>
<td>500</td>
<td></td>
</tr>
<tr>
<td>Restaurants, dining rooms</td>
<td>400</td>
<td>510</td>
<td>488</td>
<td>500</td>
<td>300</td>
</tr>
<tr>
<td>Libraries, Stock rooms</td>
<td>600/800</td>
<td>603</td>
<td>730</td>
<td>500</td>
<td>500</td>
</tr>
<tr>
<td>Garages, Passenger Cars only</td>
<td>250</td>
<td>265</td>
<td>244</td>
<td>350</td>
<td></td>
</tr>
<tr>
<td>Grand stands</td>
<td>500</td>
<td>510</td>
<td>488</td>
<td>750</td>
<td>500</td>
</tr>
<tr>
<td>Dance Halls</td>
<td>500</td>
<td>510</td>
<td>488</td>
<td>500</td>
<td></td>
</tr>
<tr>
<td>Shops</td>
<td>400</td>
<td>408</td>
<td>366</td>
<td>500</td>
<td>400</td>
</tr>
<tr>
<td>Storage warehouses:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Light</td>
<td>750</td>
<td>630</td>
<td>610</td>
<td>500</td>
<td></td>
</tr>
<tr>
<td>Heavy</td>
<td>1500</td>
<td>1000</td>
<td>1220</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Terraces: Not subjected to crowd</td>
<td>75</td>
<td>77</td>
<td></td>
<td>77</td>
<td></td>
</tr>
<tr>
<td>Subjected to crowd</td>
<td>160</td>
<td>163</td>
<td>488</td>
<td>200</td>
<td></td>
</tr>
</tbody>
</table>
2.2.2. Wind Loads: Davenport (1960) has chosen the annual extreme fast mile wind speed as the best available measure of wind for design purposes. The standard level was chosen at 30 ft. and reduction to this level by 1/7 power law was proposed for all data services. Thom (1960), Thom (1968) discussed the use of annual extremes of the fastest mile for design and presented a method of fitting the Fisher Tippett Type II distribution to a series of annual extreme winds. Davenport (1967) presented a procedure for determination of gust factors which allowed the conversion of wind speed data to design parameters. Vellozi (1968) has presented a method for calculating gust response factors and the application of the method to various examples. The calculated results of this method were presented in the form of charts which permit the direct determination of gust response factors for buildings and other conventional structures.

Dalgleish (1970) has reported on measurements of cladding pressures and in some instances compared these with boundary area wind tunnel measurements. The reliability of the gust factor approach has been established in comparison with laboratorial data by Vickery (1971) and full scale field measurements by Davenport and Dalgleish (1971). Theoretical methods of predicting response of structures to wind were reported by Davenport (1967), Vickery and Kao (1973), Novak & Davenport (1970), Waytt (1971) etc. Vickery (1970) has examined the inelastic response of structures and determined the damage rate. Davenport (1972) described the design problems of tall build-
lings, their response to wind and discussed the improved design methodologies. Ishizaki (1972)\textsuperscript{186} has introduced the well developed themes pertaining to wind loading on tall buildings: (i) meteorological properties of wind, (iii) investigation on full scale buildings, (iii) dynamic response of buildings, (iv) wind tunnel tests and (v) investigations of wind damages. Koten (1972)\textsuperscript{220} has suggested for detail investigations of: (i) wind velocities on balconies or on the roof of tall buildings, (ii) vibration of buildings perpendicular to wind direction and (iii) the problem of dynamic instability in the structure. Davenport (1972)\textsuperscript{90} has outlined the principles for more vigorous analysis of the effect of gusts by treating gusts as a stochastic (random) process. Reed et al (1972)\textsuperscript{337} have explored what a person experiences and subjectively feels in a building during a severe wind storm and suggested a quantitative measure of a person's objection to building motion.

The Indian Standard Code of Practice for Structural Safety of Buildings\textsuperscript{180} gives the details of wind loads to be considered for design of buildings. According to IS, the maximum and minimum pressures up to 30 m high above the ground are 200 and 100 kg/m\textsuperscript{2} respectively. Ramanaiah and Rao (1982)\textsuperscript{330} have analysed the wind data at some important towns and cities in Andhra Pradesh and suggested the possibility of reducing wind loads proposed in IS:875-1964.\textsuperscript{180}

ACI Committee 442,\textsuperscript{10} has reviewed and evaluated the various structural systems employed in building practice with particular reference to their function in resisting lateral
loads. Lateral load analysis and design considerations were associated with each of the systems were briefly discussed. Isolation techniques for earthquake resistance and foundations design considerations for lateral forces was briefly discussed.

Izumi (1972) made it clear that most of the variable loads act as impact and produce vibrations in structures and stressed for special considerations to minimise response motions in the structure for the comfort of the occupants and as well as safety of the structure. Sandi (1972) discussed about the probabilistic features of the most frequent types of excitations that may act on tall buildings and the problems encountered when trying to obtain reliabilities under their combined action. He stressed the importance of studying the physical processes involved as the basis for developing conceptional models of each type of excitation and then defining their stochastic models and the corresponding parameters. Vaicaitis et al (1975) have presented a Monte Carlo Method for finding the response of continuous structures to wind environment. Rojiani and Wen (1977) have presented a method for risk analysis considering uncertainties in structural strength, velocity-force relationship, turbulence and wind climate parameters. Singh and Ellingwood (1977) have proved that the most likely value of the largest of 'n' maximum yearly wind speeds is very nearly equal to the wind speed corresponding to n-year mean recurrence interval.

The proceedings of three International Conferences on wind effects on buildings and structures record the rapid development in the understanding of wind loads and the response
of the structures.

Davenport (1981) has given the methods whereby wind tunnel test results are combined with meteorological information to define design loading and the reliability of the resulting design loads.

Wen (1983) has studied the effects of wind direction on structural reliability as a problem of a vector wind force process out-crossing a structural resistance boundaries which may be direction dependant. When the structural orientation is not known or the winds have no directionality but the resistance is highly direction dependant, results from simple parametric studies indicated that the out-crossing (or failure) rate is approximately proportional to the effective "direction-window" width and there is a significant reduction in response level (or risk of failure) compared with the results based on a "worst direction" assumption.

2.2.3. Seismic Loads: Nausner (1947) has developed mathematical model treating the earthquake as a series of randomly tied half sine wave impulses. Goodman et al (1956) have suggested a velocity pulse model. Dycroft (1930) proposed to use of white noise with a particular spectral density and duration. Boltin (1960) has proposed to multiply the stationery random functions with a deterministic time dependent enveloped function to obtain a non-stationery model. Bogdanoff et al (1951) have suggested a non-stationery random process consisting of a sum of damped sinusoids representing ground
accelerations and subsequently modified by Goldberg et al (1964).

Following a procedure outlined by Lin (1960), Amin & Aung (1968) found a specific filtered non-stationary Gaussian shot noise model for earthquakes. Shinozuka and Sato (1967) have suggested two random processes to represent the ground velocity during strong motion earthquakes. The normalised autocovariances of these two processes for their stationary part compare well with those of the past earthquakes as plotted by Berstein (1930).

Bycroft et al (1950) and Ward (1965) used analogue computers to simulate earthquake motions as a stationary random process.

Rosenblueth & Bistanante (1969) have developed simple approximate solution for linear structures for the distribution of their maximum response to earthquakes idealised as segments of wide noise of equivalent duration.

Housner & Brady (1963) have given empirical equations for natural periods of vibration of buildings. An approximate technique for inelastic analysis, design and damage estimation called "Reserve Energy Technique" was given by Blume (1960). Housner & Jennings (1964) have proposed a Gaussian non-stationary filtered white noise stochastic model for generating artificial ground accelerations.

Rosenblueth (1964) has discussed the geographical distribution and annual probability of occurrence of earthquakes and the various assumptions used in the earthquake prediction and in the analysis/structural response to the earthquakes.

Benjamin (1968) has described the application of Bayesian probability concepts in forecasting probabilities of occurrence.
for earthquakes of a given intensity from historical records.

Kasiraj & Yao (1969) have presented a procedure to calculate the low cycle fatigue damage factor produced in a simple structure subjected to earthquake excitations. The variation of such damage with various frequencies differs from the variation of corresponding maximum displacements (responses) of similar structures. It is possible for a structure to fail under low cycle fatigue damage, the structural failure cannot be predicted by just studying the maximum response of a structure. In general, structures with higher frequencies (stiffer structures) are damaged to a greater extent than those with lesser frequencies by strong motion earthquakes.

Goel & Berg (1968) have concluded that the elastic fundamental period of a multi-storey structure and the general features of the elastic velocity response spectrum of the earthquake have a marked influence upon the inelastic response of the structure. The inelastic action of girders tend to decrease the response by as much as 50\% as compared with the results of the elastic analysis. Height does not seem to be a significant factor to influence the response. The assumption of the elasto plastic hysteresis behaviour tends to over-estimate the response as compared with that of a Ramberg-Osgood type hysteresis behaviour. Some corrections were proposed to account for the influence of non-stationarity and actual shape of the power spectrum by Rosenblueth & Elorduy (1969). They have suggested alternate procedures to obtain the distribution of the maximum response of stationery processes of an arbitrary and constant
power spectral density and to modify it on account of nonstationarity. Earthquake excitations are modified by some uncertain properties of the structure such as stiffness, damping yield level, dead and live loads. Hau (1968), Rosenblueth and Estava (1971) have given simplified procedures for the analysis of the influence that uncertainty about some of them has on the distribution of maximum response. Wirsching & Yao (1971) have simulated the non-stationery earthquake like motions with the use of a random signal generator and an anologue computer. Blume (1972) has stressed the need for studying biaxial response of tall buildings of slender profile.

Pinkham (1972) has reported few specific instances of earthquake damage evaluation. Osawa (1972) summarised the current state of knowledge (1972) relating to the observation of structural behaviour of buildings during earthquakes. Hisada (1972) gave general methods of seismic analysis of structures and earthquake loadings prescribed in current (1972) Seismic Codes of USA, Canada, Newzeeland, Venezuela, USSR, Japan and India. He stressed for the research needs in the field of (i) effect of interaction between the structure and sub-soil to the structural response, (ii) dynamic properties of buildings especially the damping of the higher modes of vibration and effect of secondary non-structural elements, (iii) acceptable damage level and drift limitations in relation with the type of construction and use of building etc. Jaikrishna (1972) in his discussion on "Seismic Design of Buildings" stressed that (i) the tall buildings in zones where earthquake magnitudes may
be of the order of 6.5 to 7.0 and above may be designed for the same intensity of ground motion, (ii) the columns of the tall buildings in particular in lower-half, must remain elastic under the severest earthquake and (iii) deformation, would be a design criteria for tall buildings.

In Seismic areas the structural engineer frequently encounters four different reinforced concrete column configurations that must be designed. The four types are (i) the ductile moment frame column, (ii) the captive spandrel column, (iii) the flat slab column, (iv) the pilaster column. John et al (1980) have examined the design aspects of these four types of RC column models used for analysis and detailing were considered. They have mentioned many un-resolved issues in design of strong and tough concrete columns for seismic forces.

Thompson and Park (1980) presented idealisations for the behaviour of pre-stressed, partially pre-stressed and reinforced concrete systems subjected to seismic loading and to use these idealisations to study the displacement response single degree of freedom structures to earthquake excitations using a range of earthquake acceleration records. They have concluded a good design compromise for continuous frames would be to use partially prestressed concrete beams that would combine the advantages of having some pre-stressing tendons present to balance the gravity loads and having some non-prestressed reinforcement present to increase the energy dissipation and ductility during very severe seismic loading.
Kiureghian (1981) has developed a new methodology for the reliability assessment of structural systems subjected to seismic risk. The method can be applied to any class of structural systems for which performance functions can be formulated. This may not be simple for certain structural systems such as non-linear structures for which behaviour under seismic loading is not easily described by a finite set of variables. He claimed, however, even in such cases, crude formulations of the performance function, incorporating only the most critical elements, may be assumed which will lead to more meaningful estimates of the seismic risk than the current methods. As an example he pointed out that for safety analysis of ductile R.C. structures whose failures during earthquakes are usually due to low-cycle fatigue, a simple performance function in terms of the peak ground velocity or acceleration and the duration of motion may be appropriate.

Veneziano (1981) reported the findings of the research conducted at MIT, USA, on probabilistic models of low cycle fatigue for R.C. beams and frames. He expressed the results in the form of so called "seismic fragility curves" which are relationship between the probability of failure and the value of an earthquake intensity factor such as peak ground accelerations. He reported that the Monte Carlo method used in the preliminary investigation of reliability was too time consuming.

Veneziano (1982) has briefly analysed some of the problems of seismic safety of reinforced concrete structures. He has defined the "seismic fragility" (earthquake resistance uncertainty), earthquake intensity (hazard) parameters and given
tentative suggestions for the formulation of probabilistic models of reinforced concrete member behaviour and strength and new procedures for the formulation of system reliability methods.

Krenk & Madsen (1982) have explained the classical methods of stochastic response analysis of linear structures by the evaluation of the correlation functions of excitation and determination of covariance functions of the response vis. characteristic amplitudes or failure probabilities in terms of first passage of certain barriers.

2.2.4. Fire Loads: The combustible content in a building is the potential fire hazard to both life and property. The fire safety of structural elements depends on the severity of fire it is subjected to as well as the capacity of the member to withstand such a fire. All combustible items in a building and any combustible part of a building are treated as fire loads (K.Cal/m²). The fire load is generally divided into two categories: (i) moveable content and (ii) interior finish (Bryson & Gross - 1967). Ashton and Bate (1960), Davey and Ashton, Odeen (1963), have conducted theoretical and experimental research on fire resistance of concrete structures.

The radiation or exposure hazard to nearby buildings has been studied by Law (1963), (1968). Bryson and Gross (1967) surveyed two office buildings, measured both structural and movable loads. The fire load densities were assessed as 25
and 31 kgs/m\(^2\). The portions of moveable load being 41\% and 46\% respectively. They have pointed out that the fire load is likely to increase with the time. Gross (1967)\(^{10}\) has conducted field burn out tests of apartments dwelling units. Ingberg et al (1967)\(^{175}\) have studied the combustible contents in building.

In 1970 Robertson & Gross (1970)\(^{142}\) have presented a review of fire studies in buildings. Baldwin et al (1970)\(^{28}\) have characterised the fire loads as thick cover (100 mm) thin (under 100 mm) and extended surface (wall linings, notice boards). They gave an average fire load density of 20 kgs/m\(^2\) with a skew distribution having a long tail at high values. Fire load density is shown to be independent of floor area. Thomas (1970)\(^{396}\) has carried experiments on fire in compartments of varying shape and ventilation for International Experimental Programme on Fire under the auspices of the Counsell International du Batiment. Law (1971)\(^{229}\) related effective fire resistance of a protected steel element to fire loads, ventilation area and area of bounding surfaces (walls, ceilings etc.) to which heat is lost. Fire resistance is related to fire load density modified by a factor which takes into account ventilation and size of compartment. Law (1971)\(^{229}\) has presented a simplified approach in analysing the severity of the fire in a compartment. The related fire severity to fire loads, ventilation area and area bounding surfaces to which heat is lost. ISO/R 834 (1971)\(^{190}\) has given fire resistance times of various components of buildings under standard fire condition. ISO (1972)\(^{193,194,195}\) has given the
principles of structural fire engineering design based on standard fire resistance tests.

Law & Arnault (1972) discussed the behaviour of the fully developed fire and its relationship with the standard fire used to measure the fire resistance of structural elements. They have suggested for statistical and field surveys of the effects of natural fire in tall buildings. Kim (1973) has discussed the behaviour of structures under fire, given the values of fire resistance of reinforced and prestressed concrete slabs, beams and columns and suggested to deal building fire in future design of structures like a usual design case apart from static stability, heat insulation, sound insulation etc. He pointed out the importance of repairability. He emphasised the difficulty of obtaining safe and economic design while our knowledge of behaviour in fire of structure as a whole is so limited. Petterson (1972) has discussed the approach to the fire engineering design and the economic design of fire safety and total cost of fire protection measures. He has given the characteristics of fire loads, fire growth and process of fire development.

Jensen (1972) evaluated the problem which may occur when a fire happens and the means of minimising these problems through use of protective design measures. Caliborne and Wahl (1972) have pointed out that various approaches can be followed to attain reasonable fire security but that wholesale adoption of all the features in all the existing Codes would lead to gross overdesign and suggested for urgent studies towards a better
understanding of the interaction involved. He has listed the important characteristics that are considered in the system approach to life safety from fire in tall buildings. To save the occupants of a tall building during a fire from physiologically and psychologically intolerable conditions, General Services Administrator, Public Building Service (Washington) (1972) has precisely formulated general requirements (arrangements) to move the people from areas of danger to safe areas of refuge that will not be effected by a fire conditions for the duration of fire exposure.

A brief review of fire load surveys particularly in office and residential occupancies was given by Gross (1977). He suggested the methods of measurement of fire loads and calculation of fire severity. For National Bureau of Standards (USA), Culver (1976) has prepared a report for the surveys conducted by a professional engineering firm during the period 1974-75, to determine fire loads and live loads in office buildings. From the survey data, "Room Load Intensity" and "Fire-Load Density" were evaluated.

Greene (1982) has stressed to combine the fire and live load surveys as the gathering of very little extra information such as floor covering, finishes of room surfaces, estimates of free and combined contents of rooms and the size and location of wall penetrations provides the load data needed to evaluate fire safety requirements. Ramachandran (1982) shown that the probability distribution of loss in a fire is highly skewed with long tail. Though a rare event, a very large loss could occur.
and cause widespread damage to life and property. Indian Code of Practice for fire safety of building IS:1041-1944 gives the fire resistance requirements for different structural elements composing each type of building. The occupancies are graded into three classes by the fire load content of the building. The buildings are graded into five types according to the fire load, the building is designed to resist. National Building Code of India 1970 deals with the safety from fire and like emergencies. It specifies the demarcation of fire zones, restriction of construction of buildings in each zone, classification of buildings based on occupancies, types of building construction according to fire resistance of the structural components and other restrictions and requirements necessary to minimise danger to life from fire, smoke, fumes or panic before the building can be evacuated.

Krishnamurthy (1983) suggested that a significant level of a maximum of 5% may be considered in the design of structures for fire safety. The corresponding fire load density at 95% level was 0.90 KN/m², while at 80% level it was 0.50 KN/m². The fire severity at 80% probable level was found to be 55.5 minutes, while at 95% level it was 85.6 minutes. He suggested Grade-3 or superior type of construction are suitable for high rise office buildings from safety point of view against fire hazards.

2.2.5. Blast Loads: The fundamental relations for developing blast resistant design procedures are given by ASCE (Manual No.43) (1961) and Newmark (1960, 1963), ACI Monograph
No.5 (1970). The effects of sonic boom are quite similar to those of the effects of very small explosive yields but slight difference that the positive and negative phases of the sonic boom are essentially nearly the same. Wilkins (1067) has given the effects of sonic boom on structures.

Rashbash (1969) has indicated the influence of potential explosion relief such as external windows, doors etc., in domestic structures on the pressure that may develop during gas and vapour explosions. A quantitative approach to estimate these pressures was suggested. Stretch (1969) has examined how explosions caused by vapour phased reactions between common inflammable solvents or sources of energy and air are controlled by particular features of domestic buildings and consequently strain or damage the structure during their passage, and suggested for carefully coordinated scientific experiments to allow refinement standards for economic and safe construction. Slack (1971) has studied two instances of explosions in reinforced concrete factory buildings and drawn various conclusions for possible design measures to minimise such damage.

Morales (1972) has extended the lower bound theorem of energy to transient loading problems in which loads are applied over a finite interval of time. The Theorem has been applied for illustrative purpose to blast loaded beam and plate structures. Comparison was drawn between the predictions of the lower bound energy theorem developed and the exact solution upper bound computed by Robinson (1970).
Taylor (1972)\textsuperscript{391} gave the details of work undertaken by Ballistic Research Laboratories, to improve the accuracy of the predicted load time diagram for simple structures so that more precise calculations of structural response would be made. The result of this analytical and experimental investigations show substantial differences in loading between two or three dimensional cases. A method for determining the load during the diffraction phase was presented and compared to experimental data.

Newmark (1972)\textsuperscript{289} suggested that the effects of transient loading can be related to the usual design conditions for tall buildings including either wind or earthquake design criteria to make a first estimate of the importance of the transient loading phenomena. Under special circumstances, the building can be strengthened either locally or overall in order to resist transient loading of higher intensity than the inherent capacity of the building would normally be able to survive.

Mainstone (1973)\textsuperscript{243} has reviewed various studies on measurements of pressure pulses associated with gas explosion. Action of vents in limiting the maximum pressure rise and tests on the structural response of certain types of vulnerable structural elements. Mainstone (1974)\textsuperscript{244} has reviewed the present (1974) knowledge of explosion, impact and similar hazards and the principal measures for reducing them. Mainstone (1976)\textsuperscript{245} has inspected some of the buildings in Great Britain damaged by accidental explosions, (reported incidents 2229-1976) and drawn some lessons for future guidance.
Ellingwood (1981) has pointed out that the accidental loads not presently considered may have catastrophic consequences if they occur. If the strength is not properly designed and detailed a local failure resulting from the accidental load may initiate a chain of reactions of failures throughout a major portion of the structure. He discussed the development and implementation of design procedures to control the effects of accidental loads and progressive failures.

Moore (1983) has outlined the principle features of the data on occurrence in Great Britain of various extreme incidents (explosions caused by piped gas, other explosions and vehicle impacts) for 10 years, 1971-81. He focussed on overall trends and some topical points.

Ellingwood et al (1983) have presented a case study which shows that the probability of structural failure due to a gas explosion in a residential compartment may exceed probabilities associated with unfavourable combinations of ordinary design loads and suggested to make explicit provisions to mitigate the effects of abnormal loads to be a part of building codes and standards.

2.3. Strength Analysis:

An extensive summary of statistical distribution of strength of metallic materials has been compiled by Haugen (1965). Special publication No.01-A (1963) of American Society for Testing and Materials (ASTM) gives guidelines for fatigue testing and statistical analysis of specific data.
Freudenthal and Shinozuka (1961), Stuten et al (1961), Weibul (1961), Reamsnyder (1969) have done statistical analysis of fatigue data. Freudenthal and Gumbel (1963) applied Weibull distribution to fatigue data. Dolan (1969), Gumbel (1963), Freudenthal (1968) have pointed out that hazard function, fitted with lognormal distribution, decreases with increasing life which is generally contrary to the expected fatigue behaviour of materials. Eugene (1965) found that gamma distribution function may be compatible with the fatigue phenomena. Cherez (1967) suggested a distribution function with the assumption that the growth of the crack depth can be described by a one-dimensional random walk. The resulting distribution function is complex form which provides reasonable agreement with certain available data.

The state-of-the-art in random load fatigue testing was summarised by Swanson (1968). Sweet and Kosin (1968) presented a cumulative damage theory which takes into account the randomness of the time-to-failure. Cornell (1969) with a Bayesian approach, has suggested the use of the weighted average of two distributions i.e. Weibull and gamma distributions. Wirshing and Yao (1970) have summarised and reviewed the statistical methods of estimating the parameters in the Weibull distribution as well as the confidence of these limits. Shinozuka and Itazaki (1971) analysed the fatigue failure under random loading applying Monte Carlo technique. Saunders (1972) determined confidence levels for the pro-
bability of failure as a function of the stochastic variation away from the estimate of mean life using the linear damage rule.

A series of papers by Moody et al (1954)\textsuperscript{274}, (1955)\textsuperscript{275,276}, Blister et al (1955)\textsuperscript{120} reported research on shear strength of R.C. beams, sponsored by Reinforced Concrete Research Council (RCRC). Ople and Hulsbos (1966)\textsuperscript{297} have presented a method for predicting the behaviour of concrete with a stress gradient that can be used in estimating the fatigue-life of pre-stressed concrete beams whose life is limited by concrete failure. Ang (1972), Garson and Moses (1972)\textsuperscript{145}, Wirsching and Hanger (1973)\textsuperscript{430}, Yao (1974)\textsuperscript{432} have presented simple code formats for the fatigue of structures. Sakai and Okamura (1981)\textsuperscript{302} have presented a method to evaluate fatigue damage of structural elements subjected to a stationery gaussian random loads ranging from narrow band wave to wide band one. This procedure uses the rainflow cycle counting method\textsuperscript{249} with modified Miner's linear cumulative rule.

Julian (1957)\textsuperscript{208} has presented statistical data on yield strength and ultimate strength of A-7 steel, the yield strength of intermediate grade reinforcing bars, and ultimate compressive strength of concrete. Costello and Chu (1969)\textsuperscript{80} have suggested Beta distribution for strength of steel and concrete. Alpsten (1972)\textsuperscript{3} has suggested an extreme value distribution type I or a lognormal distribution for yield strength of structural steel shapes and plates. Mirza and McGregor (1979)\textsuperscript{266} have studied the variability of mechanical properties of reinforcing bars.
They concluded that the probability distribution of ultimate strength of steel follows beta distribution. Julian (1955)\textsuperscript{268} and Shalon and Reintz (1966)\textsuperscript{305} have represented compressive strength of concrete with normal distribution and suggested that lognormal distribution gives a better fit for concrete strength in which the control is poorer (c.o.v 15-20\%). Whitehurst (1966)\textsuperscript{425} given detailed theory and application of sonic test for the evaluation of concrete properties. TSO & Selman (1970)\textsuperscript{309} have conducted field tests of concrete columns by non-destructive testing method (ultra-sonic test).

They observed that (i) there is little if any correlation between variation in cylinder strength taken on site and variation of column strength in the building. Only by direct measurements on the actual column can the variation of column strength be determined and (ii) there is a tendency that concrete at top of column is weaker than at bottom. This is observed by Peterson (1964)\textsuperscript{303}, Madhusudan Reddy (1982),\textsuperscript{242-A} ACI 214-65\textsuperscript{5} has recommended quality control charge (compression test evaluation) of moving average for strength, ranges etc. for the acceptance of concrete produced at site.

Principe (1972)\textsuperscript{315} discussed about variation of concrete strength and suggested increased control of concrete materials and concrete mixtures. Levelius (1972)\textsuperscript{231} reported a sensitive technique called CUSUM (Cumulative Sum technique) being followed by British Ready Mixed Concrete Association, for the continuous assessment of mean strength and standard deviation.
of concrete produced. Taerwe (1982)\textsuperscript{390} has given a survey of the compliance criteria for concrete strength actually in use and discussed factors which influence the operating characteristics (OC Lines) of these criterion.

ACI 316-65\textsuperscript{7} gives standards for fabricating tolerances. ACI 318-71\textsuperscript{8} specifies placing tolerances upto 0.06 depending on depth of member and bar size. Variability in workmanship is accounted by the capacity reduction factor $\phi$. CP.110; Part I, 1972\textsuperscript{31} specifies placing tolerances upto 0.08 depending on the depth of member and bar size. IS:456-1978\textsuperscript{179} specifies placing tolerances from $-6$ mm to $+12$ mm for columns and beams and tolerances for placing of reinforcement, $\pm 10$ mm for $d_{\text{eff}} < 200$ mm and $\pm 15$ mm for $d_{\text{eff}} > 200$ mm. Tolerances for cast in place concrete and precast concrete are given by ACI 347-68\textsuperscript{9}, ACI 301-72\textsuperscript{31}, PCI MNL 116-70\textsuperscript{1}, PCI Architectural Precast Concrete 1973,\textsuperscript{312} (1975),\textsuperscript{313} PCI Pub.No. MNL-121-77,\textsuperscript{290} AISC Code of Standard Practice,\textsuperscript{64} Nichols (1940),\textsuperscript{290} Connally's research report (ACI Committee 117 - tolerances 1975)\textsuperscript{70}.

Pfriang et al (1964)\textsuperscript{305} have studied the effect of column strength by varying elements of column cross section. They reported that increase in cover ratio ($d/t$) from 0.05 to 0.15 lead to a decrease in strength of column by about 10%. Black (1970)\textsuperscript{37} has reviewed the provisions of ACI 318-63, ACI 316-65 and stressed to develop compatible tolerancing standards between form work, minimum cover criteria, fabricating and placing of bars. Birkland and Westhoff (1971)\textsuperscript{36} observed that
tolerances actually obtained in construction are much larger than commonly expected and recommended a realistic tolerance of ±1″. Alpsten (1971) has reported enormous data on steel properties, dimensional variations, and residual stresses which can be anticipated in thick members commonly associated with the construction of tall buildings. Miki et al. (1971) have reported how each element of dimensional tolerance has influence on the erection of steel frames of tall buildings. Philip et al. (1973) have discussed dimensional tolerances in tall concrete buildings. Drysdale (1976) measured the locations of reinforcing bars in 232 columns in 12 structures located in Hamilton-Toronto area of Canada, concluded that bar placement errors are frequently greater than 1″ and the average bar placement error tend to result in a provision of excessive concrete cover over the exterior bars which causes decreased column capacity especially for high eccentricities of loading. Fraczek (1979) reported various concrete structural errors. Johnson (1963), Ellingwood and Ang (1972), Fiorato (1973), Mirza and McGregor (1979) have recommended normal distribution of probability models of geometric imperfections in reinforced concrete members. Giannini and Menegotto (1982) have designed 60 statically determinate R.C. slender columns according to CEB Model Code and tested, and confirmed that Model Code rules are quite fitted for designing elements of uniform safety with regard to the slenderness and to some extent to the load.
intensity and eccentricity. Kuczynski and Tkačovský (1982) have reported experimental results and as well as a theoretical analysis based on the so called continuous theory of bending of reinforced concrete. They have compared experimental data with theoretical analysis and given some recommendations.

Lima (1982) has presented some of the basic problems of brittle materials like concrete which are susceptible to catastrophic failures. He discussed the concept of structural ductility, its significance and approach to quantify it. Lima & Lima (1982) have tested beams in flexure with various percentage of tensile reinforcement. In a beam subjected to bending moment, if the age of tensile reinforcement is continuously reduced, one finds different mechanisms of failures, each one of them more brittle than the previous one. They have reported the main characteristics of these mechanisms.

2.4. Dynamic Structural Analysis

the mean square response and the expected frequency of zero crossings for the random vibration problem in a non-linear system with a setup spring and compared with approximate solutions using equivalent linearisation techniques. The probability density functions for the peaks and the response envelop were obtained to study the probability of failure of the systems. Rosenblueth and Bustamante (1962) have studied the distribution of peak responses to earthquake type random excitations.

Caughey (1963) derived and applied the Fokker-Plank equation to discrete non-linear dynamic systems subjected to white noise excitations. Crandall (1963) computed probabilities of zero crossings, threshold crossings and distribution of peak from the joint density function of the response displacement and velocity of a non-linear system subjected to random excitation. Crandall (1963) in a subsequent paper described and illustrated the application of the classical perturbation method to obtain the random response of dynamic systems which are slightly non-linear. It was found that the effects of a hardening spring tends to increase the expected frequency of zero crossings as well as to decrease the amplitude of the peaks of the response of the system, as compared with the corresponding linear system. Caughey (1963) in an alternative approach, generalised the method of equivalent linearisation to solve the problem of non-linear dynamic systems with random excitation. Lin (1967) has summarised and described all these methods.
Goldberg et al (1964)\textsuperscript{160} studied the statistics of the response of simple non-linear systems in a large number of earthquake excitation generated with a digital computer. They found that the addition of hard non-linear springs may provide substantial protection against seismic disturbances. Goel and Berg (1968)\textsuperscript{149} have studied the non-linear, elasto-plastic response of multi-storey frames under seismic loads.

Simiu (1974)\textsuperscript{374} presented improved expressions for longitudinal wind spectra, which were consistent with recent theoretical and experimental results of boundary layer meteorology, in which the variation of wind spectra with height was taken into account. Expressions for the alongwind response including deflections and accelerations which take into account higher vibration modes were proposed.

Joseph and Radhakrishnan (1976)\textsuperscript{206} have developed frequency characteristic chart which provides good trial values of the intermediate frequencies for adoption in the Holzer technique or the modified Holzer iteration techniques. A non-dimensional parameter termed as "Nature Parameter" connecting the fundamental and highest frequencies and the total number of floors was explained, which depicts in a better manner the buildings inherent behaviour in vibrations. The expression for a non-dimensional parameter called "Period Parameter" connecting the period, total stiffness (sum of storey stiffnesses) and total mass (sum of the storey masses) was defined and the associated tables were given, which act as simple design tools in deter-
mining the natural frequencies and mode shapes of shear buildings.

Shinozuka (1974) emphasized that although mathematical tools to model natural phenomena for the purpose of performing a dynamic safety analysis appears to be adequate but much further effort is needed to establish a methodology to consistently deal with insufficient knowledge and the data and the inherent uncertainty associated with dynamic safety analysis.

Roorda (1975) has examined some of the concepts involved in active damping of structures by feed-back control and amplified by way of analysis of a tendon control scheme for tall flexible structures (masts and towers etc.). The analysis was for a single mode vibration. Results showed an effective attenuation of vibration response in first mode. He suggested for the study of interaction between modes and the real possibility of the \( m \)th mode active damper causing an instability.

2.5. Structural Design and Reliability Analysis:

The basic concept of Structural Reliability Analysis was first outlined by Freudenthal, [133,134,135,136,137,139] Pugsley (1951), 316 (1959), Brown (1960), 49 to derive the minimum safety factors from statistical variation of the design characteristics. Zutty (1963) 440 presented a prediction equation for ultimate moment of under reinforced concrete beam. Shah (1964) 364 developed prediction formula for ultimate column loads based on regression analysis of tests on short tide columns conducted in U.S. since 1930. Shinosuka (1964) 369 estimated the probability of failure of a simple structure.
caused by a simple application of random input of known intensity. Cornell (1967)\textsuperscript{71} gave lower and upper bounds of structural reliability considering various degrees of statistical independence of load and resistance. Turkstra (1967)\textsuperscript{400} has formulated a generalised basic scheme for the choice of safety level in structural design for certain conditions of failure, on the basis of maximum expected loss criteria. Ang & Amin (1968)\textsuperscript{15} have established a monotonic property for hazard function of structures. Costa (1968)\textsuperscript{79} criticised the attitude of most of the engineers (1968) for their reluctance in accepting the probabilistic concept of structural safety pointed out how redundant elements can contribute to increase the overall safety of a structure. Lind (1968)\textsuperscript{237} has estimated the amount of data required to make confidence interval statements about the probability of failure by methods of mathematical statistics.

Warner & Kabaliya (1968)\textsuperscript{417} have described Monte Carlo technique to determine the cumulative distribution of stochastic variables. The ultimate strength of an isolated short reinforced concrete column under axial loading was investigated assuming rectangular uniform density functions for variations in concrete strength, yield strength of steel and concrete area. Benjamin (1968)\textsuperscript{31} described some of the advantages of rational probabilistic analysis. Sexsmith (1969)\textsuperscript{361} presented procedures for determining probability distribution of safety margin and related quantities of different reinforced concrete members. In a series of articles by Shah (1969), Sexsmith and

Allen (1970) studied the bending of R.C. members and concluded that (i) there is a significant probability that a section, under reinforced according to ACI 318-63, will undergo a brittle compressive failure, (ii) the variability of ultimate moment expected in practice increases either when the member is thin or when the percentage of steel is high. This increase can be reduced considerably by good workmanship, (iii) the variability of ductility ratio is much higher than the variability of ultimate moment. Douma (1971) suggested the responsibilities of Engineers & Contractors involved in various stages of design and construction. Johnson et al (1971) have explored and discussed boundaries of responsibilities. Attention was focussed primarily on the frictional interface between design and construction teams and suggested remedies. Lind (1971) showed how partial safety factors in a design code can be selected to balance safety and economy. He compared the rationale of various code formats vis. Ang, Amin (1969),
Benjamin & Lind (1969), Cornell (1969), ISO Draft (1969), Optimization (Lind) (1969). Ang & Ellingwood (1971) have suggested "extended reliability concept" for handling imperfect information about applicable probability distribution of the random variables defining design as well as uncertainty stemming from sources for which probabilities cannot be inferred from statistical analysis of repeated experiments. Turkstra & Shah (1972) have pointed that the parameters defining uncertainty are uncertain themselves.

Moses & Tichy (1972) grouped system reliability models into two categories: (1) weakest link model and (2) ductile mechanism model (collapse model). In Weakest Link Model it is assumed that the system fails if any of its member fails. The Ductile Mechanism Model assumed that stress re-distribution takes place each time that the yielding limit is reached at a critical section that a new re-distribution occurs again, as the load is increased and yielding starts at new critical sections and that an unlimited capacity of plastic deformation is available at each section. Collapse occurs when the number and location of yielding section is such that a mechanism is formed.

Ang (1973) described rational probabilistic analysis and design concepts in comparison with existing deterministic procedures. Ang & Cornell (1974) have established that design criteria may be formulated most rationally within a probabilistic framework. In order to accomplish this, methods were developed for analysing design uncertainties systemati-
ally and a model was formulated for evaluating the risks associated with various designs. Ellingwood & Ang (1974) have illustrated the quantitative analysis of design uncertainties and shown how these uncertainties affect the level of risk. The risk associated with existing design procedures were evaluated with reference to reinforced concrete. Ravindra & Lind (1974) have illustrated the application of a probabilistic design method - the safety index method by means of a variety of practical examples.

Moses (1974) has pointed out that the area of reliability of structural systems is not as well developed to date (1974) as that of reliability of elements. He has presented some simplified analysis approximations including the finding of appropriate partial safety factors for elements in systems.

Melink (1974) has described some methods of optimising the level of safety expenditure and discussed the advantages of putting a value on each life saved. Some methods of obtaining the value of life were outlined.

Hasofer & Lind (1974) have presented a reliability analysis using second order moments. The format is fully invariant. It reports reliability in terms of safety index, \( \beta \), and makes use only of the first and second statistical moments of the uncertainty vector. It is a discrete point checking method measuring the minimum distance \( \beta \), between the boundary of the safe domain and the means of the uncertainty vector in terms of standard deviations of the functions.
the limit state. Palchiamo & Hannus (1974) have introduced a relatively simple "Invariant" safety index structural design method. A deterministic design equation is derived in which all random variables are substituted by appropriate design values. A design value is a function of the mean and standard deviation of the corresponding random variable.

Ellingwood (1975) has summarised the procedure for the first order second moment reliability analysis. Measures of safety that are tacitly associated with American Concrete Institute Standard 318-77 were also computed in order to establish bench marks for the subsequent development of reliability based design criteria. Wirsching (1975) described the errors introduced in the most widely used approximate method for calculations. The truncation errors tend to be greater with skew distributions because of the existence of the third central moment.

Chandrasekhar & Dayaratnam (1975) presented the reliability analysis of simply supported pre-stressed concrete beams. Probability of failure was evaluated based on the governing equation of the ultimate strength of the section fixed by deterministic analysis assuming normal distribution for the resistance of the section. Drysdale (1975) has studied the distribution of bar (longitudinal reinforcement) placement errors in concrete columns. He assumed normal distribution and found the reduction in column capacities for high eccentricities of loading.

Lind (1977) has extended the safety index approach of
second moment statistical design to design for specified reliability by a transformation of the random variables to a standard Gaussian set. Kunamoto et al (1977)\textsuperscript{224} have presented a new Monte Carlo method to estimate the reliability of a large complex system represented by a reliability block diagram or by a fault tree. Large number of trials are required to obtain reasonably precise estimate of the reliability when crude (straight forward) Monte Carlo method is used. A better Monte Carlo method is developed by applying variance reducing techniques. It is claimed that the new Monte Carlo method gives a better reliability estimate with a smaller variance than that of the crude Monte Carlo method.

Fujino & Lind (1977)\textsuperscript{142} have studied for the practical interpretation of proof load tests as a design tool, utilising second moment reliability concepts, towards implementation of the "\( \beta \) - method", in test procedures. Ellingwood (1977)\textsuperscript{114} studied resistance of RC beam column by Monte Carlo technique assuming normal distribution for variation in strength of concrete and geometric variations and lognormal distribution for variation in yield strength of steel, for various degrees of quality control and workmanship. Grant et al (1978)\textsuperscript{153} have studied the ultimate strength of rectangular reinforced concrete short columns assuming normal distribution for strength of concrete, geometric variations and reinforcement locations and beta distribution for yield strength of reinforcing bars. Ranganathan and Dayaratnam (1978)\textsuperscript{333} presented a method for evaluation of reliability of PSC flanged sections. They observed that there is a possibility
of PSC section becoming over reinforced even though the section is under reinforced deterministically. Even though this probability is small it should be considered in the evaluation of probability failure of a section. Bury (1978)\textsuperscript{53} presented a simple design problem to illustrate the relationship between product reliability and design strength for the case where product failure is due to a single overload in a sequence of loads applied during the mission of the product. Bosshard (1979)\textsuperscript{48} has reviewed the two new tools of structural reliability, i.e. (i) statistical prediction modes and (ii) second moment reliability. An attempt was made to redefine certain fundamental concepts of structural safety in a more rational way. Ellingwood (1979)\textsuperscript{116} has illustrated the selection of practical ultimate limit state criteria for reinforced concrete design.

Pugsley (1973)\textsuperscript{318,319} discussed the likelihood of errors causing structural accidents, whether involving partial or total collapse occur either because an error is made in the design or construction of the structure or because several unfortunate circumstances just happened to combine. He attempted to isolate the pressures on the designer and contractor in order to predict whether or not the structures will be prone to an accident. He discussed the pressure under the headings of political, financial, scientific, professional and industrial climate and identified 8 important parameters. Attempts have been made by Blockley (1973)\textsuperscript{38} to formulate ways of assessing the magnitude of these parameters. Blockley (1975)\textsuperscript{40} has outlined a procedure for
predicting the likelihood of a structure failing due to causes other than the stochastic variations in loads and strengths, using the concept of Fuzzy sets. Such a method can use arbitrarily yet precisely defined linguistic variables which are suitable for subjective estimation. Sadegh (1966), (1973), discussed in detail this new approach to the analysis of complex systems. Blockley (1972) has reviewed 23 major structural accidents and analysed using a simple numerical interpretation. The accidents were ranked in their order of inevitability. A simplified form for predicting the likelihood of structural accidents was outlined using the concept of fuzzy sets. Yao (1979) has explained the method of application of fuzzy sets in the assessment of fatigue of fracture reliability.

Brown (1979) presented the calculus for obtaining a safety measure that includes both subjective and objective informations. The objective part includes all countable information that can be averaged or limited. Objective probabilities are obtained through the principle of maximum entropy. This ensures that the probabilities obtained are unbiased and personally invariant. The subjective part includes all other information that cannot be expressed in such an objective form. The subjective part has no countable constituent and must be conceived as a mental abstraction that can be expressed linguistically.

Blockley (1981) has discussed the underlying types of uncertainty and outlined the limitations of probability. A
method of combining fuzzy propositions with associated dependability values in a hierarchy were described and applied to the problem of structural safety.

Ditlevsen (1983)\textsuperscript{101} has discussed basic concepts of fuzzy set theory (or Fuzzylogies). He has argued for expressing structural safety in terms of a pair of numbers ($P_{th}$ & $P_{gr}$) rather than by a single number, where $P_{th}$ is the theoretical failure probability as it results from the structural reliability theory disregarding the existence of gross errors & $P_{gr}$ is the measure of proneness to failure due to gross errors in terms of fuzzy sets. Brown et al (1983)\textsuperscript{51} have examined the simple theory and applications of Fuzzy sets. The application of theory allows the inclusion of professional wisdom, knowledge and intuition that into an analytical scheme. It has been argued the probability theory and statistics are useful in Civil Engineering but their use is limited in the sense that most civil engineering decisions are made with a shortage of numerical evidence and depend on informed opinions. The fuzzy set theory is intended to deal with the informed opinion, but in no way tends to disperse with countable evidence. Yao (1979)\textsuperscript{436} has summarised and reviewed the available literature on safety evaluation of existing structures and discussed several approaches and formulations of problem including damage functions, pattern recognition and fuzzy sets along with the analysis of available data. He suggested to continue the efforts to examine the problem of damage assessment in the context of pattern recognition. Rockwitz &
Fiessler (1978)\textsuperscript{326} have presented a technique to compute the reliability of structures under combined loading assuming that loads and other actions can be modelled by independent stationary random sequences. Garson (1980)\textsuperscript{144} has developed an approximate method of system risk analysis of Weakest Link Structural System (a weakest system is one that is said to have failed whenever any of its member has failed).

Biernatowski (1981)\textsuperscript{35} has discussed the possibilities of the theory of safety and reliability of soil structure interaction. He formulated and discussed mathematical models of reliability and the structures in parallel in series, and in series in parallel.

Klingmuller (1981)\textsuperscript{217} has pointed out that the idea of redundancy as a source of safety, which is quite common in system reliability cannot be directly applied to structural redundancy. The analysis of structural safety relies heavily on the difficulty of mechanical modelling of structural behaviour and so the influence of redundancy which in structures is given as 'hot' or 'standby' redundancy is not exactly determinable. Some approaches were shown and suggested further research into the effect of redundancy in structural reliability before redundancy can be actually used in a "design for safety".

Handa and Anderson (1981)\textsuperscript{159} have presented a finite element technique for the estimation of mean values, standard deviations and correlation coefficients of structural displacements and stresses taking into account variations in applied
loads, dimensions and material properties, with the increase of the number of elements the method can more and more accurately account for the non-homogenity.

A general probability distribution transformation has been developed by Hohenbichler & Rockwitz (1981)\(^{163}\) with which complex structural reliability problems involving non-normal, dependent uncertainty vectors can be reduced to the standard case of first order reliability i.e. the problem of determining the failure probability or the reliability index in the space of independent, standard normal variates. The method requires the knowledge of the joint cumulative distribution function or a certain set of conditional distribution function of the original vector. Cornell (1981)\(^{75}\) has given the major examples of development in the last 20 years in 2 specific areas of probability based structural building codes and seismic safety of Nuclear Power Plants. He observed that 'Structural Safety as a whole evolved well in the last decade and that it should continue to gain strength in the next'.

Ang & MA (1981)\(^{20}\) have identified the problems of reliability of structural systems against initial damage and collapse and reviewed the current methods for estimating the system collapse probability and its bounds. The difference between the reliability problems of ductile and brittle systems were highlighted. The application of PNET\(^{17}\) method of ductile system was described and illustrated.

Murotsu et al (1981)\(^{282}\) have presented a method for generat-
ing failure modes of redundant structures by using matrix method and the reliability (assuming normal distributions for strengths and loads) assessed by evaluating upper and lower bounds of the structural failure probabilities. A method was proposed which calculates the bounds by selecting the dominant modes of failure. The validity of the proposed method was demonstrated with numerical examples. Muotsu et al.\textsuperscript{283,284} have developed method for assessment of reliability (for the case of non-normal distributions of strengths and loads) by combining the method of selecting dominant failure modes and the conventional method of reliability analysis i.e. the method of calculating the structural reliability based on the assumed modes of failures.

Konishi & Takaoka (1981)\textsuperscript{219} have carried the reliability analysis of compression members using the theory of random functions. They have concluded that the eccentricity of the axial load has considerable influence on the probability of failure for shorter column range while the variability of initial deflection exerts its influence on the probability of failure in the slender column range.

Schuller (1981)\textsuperscript{356} reviewed the different reliability methods i.e. binary approach, response surface method, simulation method etc. He investigated the applicability of these methods for the reliability analysis of various types of systems particularly with reference to their capabilities of reflecting the actual physical process. He has studied some other aspects of the reliability analysis of nuclear power
plants. For the overall reliability of Nuclear Plant System, electronic, mechanical and structural systems have to be analysed. It was shown that this requires particular attention of the interactive effects between their various types of systems. He suggested for the collection of data in the field of macro-seismicity and further investigations of the behaviour of R.C. Structures under impact loading.

A current International view (1981) on the state of the art of the reliability analysis of nuclear power plants were presented during the 3rd International Seminar on Reliability of Nuclear Power Plants at Paris.\textsuperscript{32}

Extensive reliability studies were conducted by National Centre for Systems Reliability (NCSR),\textsuperscript{63} on behalf of Atomic Energy Authority, Great Britain, related to electronic and electro-mechanical systems and reactor containment structures. In assessing acceptable risks, NCSR studied medical and accident risks implied in other fields and have formed measures of levels of security generally of an order of magnitude higher than commonly intrinsic in design. This enhanced reliability is considered desirable in view of political (and hence economic) consequences of a structural incident.

Ditlevsen (1982)\textsuperscript{102,103} reviewed the basic reliability concepts and the role of crossing theory in structural analysis. He presented details of crossing theory that are of particular interest in reliability theory.

Rockwitz (1982)\textsuperscript{323} presented so called first order reliabi-
lity methods, summarised the most important types of structural failure, their definitions in a decision theoretic framework and certain implications on the modelling aspects, reviewed the basic methodology for component reliability and analysed the structural systems. He presented a methodology to calculate the probability content of arbitrary domains occurring in the reliability analysis of structural and other technical systems. Rockwitz (1982) has reviewed the present (1982) status of first order reliability methods. He has pointed out that the choices of the type and number of basic uncertainty variables and the corresponding stochastic models are not unique and, therefore, the outcome of reliability calculations or of optimisation is ambiguous. It appears useful to limit the number of variables i.e. pool less important ones into certain representative variables and limit the class of admissible stochastic models as far as possible without loosing too much flexibility if new data becomes available.

Konig & Hassofer (1982) have shown that using simplified level II method (as described later in Chapter 3), the deviation from the target reliability, given by a safety index $\beta$, are within acceptable bounds for the most practical design situations. They have discussed some of the aspects of appropriate design formats for codes, desirable and acceptable simplifications especially in load combinations.

Lind (1982) has outlined the concept of structural quality used to represent structural reliability combined with
low maintenance and few repairs and suggested that the sum of optimal structural repair and maintenance cost as a basic measure of structural quality. The quality expectation has been expressed as a volume integral in load-time-space. He suggested to develop practical methods of structural quality assurance that are reconcilable with a common philosophy of structural quality.

A reliability analysis approach for structural systems taking into consideration the deterioration of the resistance due to random loading effects was developed by Oswald & Schmoller (1982). They have described the functional relation between the deterioration of the resistance and the loading history. The limit load range was included in order to treat the physical process consistently over the entire range of the expected load spectrum. This approach unlike the method of linear damage accumulation based on S.N. curves, allow the consideration of the influence of the initial condition of the material i.e. the number and the severity of the flaws present in the materials i.e. produced by the fabrication process.

The use of probability of failure as a criterion was discussed by Jordaan (1982) using 'unility theory'. He has pointed out that there is scope for further explicit consideration of the balance between cost & safety that is implied in the utility function.

Mathiew (1982) has presented a study on 'non-exceedance conditions for limit states' in the form of codified tentative clauses, as a possible extension of the corresponding part of
the 3rd draft of Eurocode 1.

Karamchandani (1982)\textsuperscript{10} has classified failures according to significance of defects. Causes of failures were identified due to faults in design, execution, materials used and unexpected user requirements. The failures, on investigation and analysis were found to be caused by any one of these causes either singly or collectively.

Kam et al (1983)\textsuperscript{209} have presented techniques for approximating the limit state equations for two and three random variables and methods for evaluating failure probabilities. A general expression was given in numerical integration form for computing failure probabilities with multiple variables.

The Committee on Reliability of Offshore Structures (1983) has brought a perspective on current applications of reliability methods in the analysis of Offshore Platforms and developed design guidelines. Advantages and dis-advantages of present reliability methods were discussed and the need for and direction of failure research that will strengthen existing methods was outlined.

Kelchers (1983)\textsuperscript{257} concluded that for structures which can be modelled in a reliability analysis as parallel structures, the effect of high member strength can be of significance in estimating system reliability. It was shown by means of examples of simple frames that the reliability of the system is sensitive mainly to correlation between the member strengths within the one-member or between related members.
2.6. **Ergonomics (Human Reliability)**

The branch of applied experimental psychology now known as "Ergonomics" or 'Human Factors Engineering' is heavily endowed with engineering philosophy and terminology. It is largely concerned with man-machine interaction tasks, such as gauge reading, dial setting, adjustment and visual interpretation of displays. In a broad sense the functions of interest are psychomotor, monitoring, controlling and visual inspection.

Principal applications of ergonomics has been in the military area, and also more recently, in the nuclear industry and heavy process industries such as chemical engineering. Application has also been concerned almost exclusively with machine operation, study of the higher cognitive tasks, such as one involved in the design process, is almost non-existent. 121

Meister (1966) 266 gave a useful over view and some typical results. Most studies report only point estimates rather than frequency distributions or second moment estimates of reliability. There is also a not inconsiderable problem with experimental design, it being difficult to replicate real situations in the laboratory. 386

Swain et al (1979) have performed an extensive human error analysis for a loss of coolant accident (LOCA) as a result of valve operation in a nuclear power station. Similar work has been reported by Okrent. 296 The latter studies were based on the use of 'fault trees' requiring division of tasks into the binary subject 'successful' and 'un-successful'. 
Rockwitz (1977) has tried to account for human errors and negligence by superposing such errors as additional random effects on the models of structural reliability.

Human reliability information is also required in establishing the relationship between human error made in a design task and its manifestation as an error or defect in the structure. Human error is an important component in proper reliability assessment, whether it be for absolute structural reliability calculation or for the relative evaluations pertaining to code calibration. Melchers (1978), (1982) reviewed and reported the work on human errors in structural engineering design related tasks and indicated how such an error assessments can be readily incorporated into existing limit-state type formulations for assessing second moment (or higher order) reliability.

Smith (1976); (1977) found that bridge failures to a large extent had natural causes while Matoušek (1977) found that almost all structural failures were due to human errors. Human error is apparently an important source of loss in structures. Lind (1982) has developed the notion of 'human error' as a cause of system failure, presented models of human error influence on structural reliability and the approach to structural error control.

Reij & Toorn (1982) have emphasized the rate of gross human error in a reliability analysis and discussed the necessity to incorporate this in the analysis. A case history was presented and discussed and the following lessons were drawn:
(i) despite extensive precautions, some error can always occur, (ii) also the use of fault and event tree (though very useful in avoiding gross human errors in design and construction) cannot guarantee that no error will occur, because by its very nature this technique is not suitable to detect errors, caused by a non-imaginable combinations of un-expected events. He stressed for adequate control and inspection system on site to cover these inherent shortcomings.

Matousek (1981)\textsuperscript{251} stressed for a new overall safety concept including strategies to avoid and/or to detect and correct human errors. The efficiency of these strategies within the safety concept should be checked in future by an appropriate failure analysis.

2.7. Structural Codes :

The Western and Asian Codes on structural analysis and safety are more or less based on limit state theory and on the other hand the approach of USSR and other socialist countries is different.

CP 110:Part I, 1972, IS:456-1978\textsuperscript{81} and IS:179 are based on limit state design. ACI 318-77\textsuperscript{8} is based on strength design approach. A sub-committee (E) was formed by ACI Committee 348, 1978\textsuperscript{386} for giving proposals for probabilistic code format. A IS-1850\textsuperscript{11} has developed load criterion, including load factors and load combinations suitable for limit state design.

The new unified computational methods came into force in USSR in the year 1955 for civil and Industrial constructions.
of Reinforced Concrete. (SNIP II-B, 1-62). The socialistic countries adopted this computational method (in the year 1963) based on the recommendations of the International Council for Reciprocal Economic Aid (COMECON). This method was introduced in Romania in the same year in the form of a conditional normative and latter definitised under the ‘STAS 800-67’ (Ultimate Stress method).

Stiller (1972) compared the then existing codes of practice in European Countries viz. Finland, Sweden, Norway, Denmark, Netherlands, Belgium, Luxembourg, France, Great Britain, Greece, Italy, Spain and Portugal based on a questionnaire sent to the representatives of the above countries.

The concept of safety clause is implemented in USSR in 1981, in a very general frame-work based on the social benefits and reducing the consumption of material, (Ostanover et al., Nordiac countries base the concept of safety clause on the consequences of failure, risk to life and social consequences. (The Nordic Committee for Building Regulations, Copenhagen, 1978).