B3.1 Detailing for Hurricanes

by Tony Gibbs, BSc, DCT(Leeds), FICE, FIStructE, FASCE, FConsE, FRSA
November 2000

1 General Introduction to Wind Damage and Design Against Wind

1.1_ Hurricanes in the Caribbean

In recent times in the Caribbean there has been a heightened awareness of the damage potential of hurricanes. Such damage is done both by wind and by water. Damage by the wind can be either direct (*ie* due to wind pressures acting on the structures) or indirect, as in the case of flying debris impacting on vulnerable structures such as glazed windows.

The pattern in recent times has been a reduction of deaths and injuries (because of better warning systems and other preparedness activities) and an increase in property damage (because of commercially-driven unsuitable building practices and locations).

Anyone who actually experiences a major hurricane, especially during daylight hours, would realise how inadequate and sterile are wind-loading standards. A quotation from Professor Joseph Minor, modified by Tony Gibbs, puts it this way:

> *"The real environment in a hurricane consists of strong, turbulent, winds (sustained for many hours), that change slowly in direction as the storm passes, and carry large amounts of debris while accompanied by torrential rains."*

Hurricane straps and other wind-resistant devices should be tested recognising the characteristics of repeated, dynamic and fluctuating loads in such events.

During the past twenty years there have been several memorable hurricanes in the Caribbean. These events repeatedly illustrated the effects of such storms on buildings and other structures. They include Hurricane David in the Commonwealth of Dominica and the Dominican Republic in 1979; Hurricane Gilbert in Jamaica and Mexico (Cancun, Yucatan) in 1988; Hurricane Hugo in Dominica, Guadeloupe, Montserrat, Antigua, St Kitts, Nevis and The Virgin Islands in 1989; Hurricane Andrew in Cat Cay in the Bahamas in 1992; Hurricane Luis in Antigua and Sint Maarten / St Martin and Hurricane Marilyn in St Thomas in 1995, Georges in Antigua-Barbuda and StKitts-Nevis, Floyd in The Bahamas, Lenny in the north-east Caribbean.

1.2 Damage due to Hurricane Winds

The main types of damage to buildings and structures experienced in Caribbean hurricanes are:

Foundations — The uplift forces from hurricane winds can sometimes pull buildings completely out of the ground. In contrast to designing for gravity loads, the lighter the building the larger (or heavier) the foundation needs to be in hurricane-resistant design. Anchoring buildings to their foundations is critical for lightweight structures.

Structural Frames — A common misconception is that the loss of cladding relieves the loads from building frameworks. There are common circumstances where the opposite is the case and where the wind loads on the structural frame increase substantially with the loss of cladding.

Usually the weakness in structural frames is in the connections.

Masonry Houses — These are usually regarded as being safe in hurricanes. There are countless examples where the loss of roofs has triggered the total destruction of un-reinforced masonry walls.

Timber Houses — The key to safe construction of timber houses is in the connection details. The inherent vulnerability of light-weight timber houses, coupled with poor connections, is a dangerous combination which has often led to disaster.

Roof Sheeting — This is perhaps the commonest area of failure in hurricanes. The causes are usually inadequate fastening devices, inadequate sheet thickness and insufficient frequencies of fasteners in the known areas of greater wind suction. (See OHPT[1]-1)

Rafters — Of particular interest in recent hurricanes was the longitudinal splitting of rafters with the top halves disappearing and leaving the bottom halves in place. The splitting would propagate from holes drilled horizontally through the rafters to receive holding-down steel rods.

1.3 Prevention of Damage

Hurricanes are not natural disasters, they are natural events which sometimes lead to manmade disasters. In these days of widespread technological education, sophisticated research, reliable building materials, computer-based geographical information systems and satellite-assisted warning programmes, hurricanes in the Caribbean should not lead to disasters.

[1] Overhead projector transparency

One of the first steps which must be taken by the construction industry is the use of appropriate standards addressing the effects of wind on buildings.
Apart from formal and mandated standards, the most effective influence on the improvement of the security of buildings against hurricanes can be wielded by the general insurance industry. Insurance companies have a vested interest in this subject and could provide a strong incentive for the improvement of standards of design and construction.

Most insurance companies provide hurricane cover at the same rates for most buildings, irrespective of their relative abilities to withstand natural hazards. In this system "Peter pays for Paul". Graduated premiums, based on design type, materials and quality of construction would be a meaningful step in the right direction. There is some evidence that Caribbean insurance companies are moving in this direction.

So what can be done for new construction as well as for the large existing stock of buildings? Quite a lot. Listed below, in very general terms, are some issues which should be addressed for new construction and then for the strengthening of existing buildings.

Location	The location of the building is important. We often have little choice in the matter, perhaps because of financial constraints. It is as well, therefore, to recognise when a building is being located in a more vulnerable area. The rational response would be to build a stronger-than-normal house. Such vulnerable areas include open-ended valleys, which act as funnels for the wind, and exposed hill crests. Both conditions lead to acceleration of wind speeds with the corresponding increase in damage potential.
Shape	We do have control over the shape of new buildings and shape is the most important single factor in determining the performance of buildings in hurricanes. Simple, compact, symmetrical shapes are best. The square plan is better than the rectangle. The rectangle is better than the **L**-shaped plan. This is not to say that all buildings must be square. But it is to say that one must be aware of the implications of design decisions and take appropriate action to counter negative features. Even more important than plan shape is roof geometry. For lightweight roofs it is best that they be of hipped shape (sloping in all four directions, usually), steeply pitched (30 to 40 degrees), with little or no overhangs at the eaves (with parapets if possible) and with ridge ventilators where these are practicable. (See OHPT-2.)
Materials	The strengths of materials are important characteristics as would be expected. Durability is equally important, especially in the corrosive environments prevalent in coastal situations which are commonplace in all Caribbean islands.

Forces Although the determination of wind forces on buildings is not a precise exercise, it is nevertheless desirable to use the information in standards documents such as CUBiC and BNS CP28 to get better approximations of the forces and the patterns of forces than mere guesswork can provide.

Windows and Doors Apart from roofs, the elements requiring the most attention are windows and doors. Sadly, these are often neglected, even when buildings are formally designed by professionals. Glass windows and doors are, of course, very vulnerable to flying objects. And there is much flying debris in hurricanes.

Connection Details The famous German architect, Mies van der Rohe, used to preach to his students that "God is in the details". For anti-hurricane construction this could be rewritten "God is in the connections". It is imperative that all the components of a building envelope be securely interconnected.

Retrofitting We must also address the huge stock of existing buildings. Any improvement is worthwhile. It won't be easy (it may not even be possible) to protect many existing buildings from major damage in another David, Gilbert, Hugo or Andrew. But all hurricanes are not great ones. The more severe the storm the less frequent its occurrence. Conversely, the less strong the hurricane the greater the likelihood of it visiting any particular community. Small improvements would be needed more frequently than major strengthening so, at least, a start should be made with the small things.

Add to and improve the connections of lightweight roofs to purlins, purlins to rafters and rafters to walls; invest in storm shutters; add bolts to external doors; increase the connections of door and window frames to walls; pay attention to the maintenance of buildings.

Costs The good news is that using hurricane straps is affordable to all building owners.

1.4 Reinforced Concrete and Steel Frames

The detailing requirements for reinforced concrete and steel frames for wind resistance are essentially the same as for gravity loads except, of course, for the directions of the loads and the corresponding distributions of moments, shears and direct forces. Therefore, nothing further will be said here about this issue.

1.5 Masonry

Masonry, whether load-bearing or not, located on the exterior of buildings must be detailed so as to resist in-plane forces and forces transferred to the wall by the building as a whole and by the direct

wind forces normal to the surface of the wall. The latter forces should be catered for by horizontal and vertical reinforcement obtained from a rational analysis of the wall panel.
In several countries (including South Florida) the practice is to secure wall panels by means of stiffeners (or columns) placed at intervals around the perimeter of the building. It is very easy to demonstrate by analysis that this is often an unsatisfactory solution to the problem. The proof of this is the many unreinforced wall panels that are damaged and destroyed in major hurricanes even when surrounded by stiffeners and belt beams.

1.6 Glazing

Impact-resistant glazing is slowly gaining popularity as a loss-reduction strategy, even for speculative construction. It involves the use (most commonly) of laminated[2] glass fixed into frames with structural silicone. This is a case of the weakest link syndrome. The glass, the fixing cement, the frame and the fixing of the frame to the structure must all be satisfactory for the system to succeed.

1.7 Roof Coverings

The typical light-weight roof coverings in use in the Caribbean are:

- arc-tangent profiled metal sheets;
- trapezoidal profiled metal sheets;
- various membrane waterproofing materials on close-boarding;
- barrel tiles of clay or concrete.

After Cyclone Tracy[3] a "cyclone washer" was developed in Australia (OHPT-3) which is an effective fastening system for *arc-tangent profiled metal sheets*.

In the French Antilles variations on that product are used for both *arc-tangent profiled metal sheets and trapezoidal profiled metal sheets* (OHPT-4).

The range of *membrane waterproofing* materials is too large to provide any specific guidance on fixing methods. They usually fall into the categories of cements or nails and sometimes a combination of the two systems.

[2] The laminate is usually polyvinyl butryal (PVB)

[3] Darwin, Australia, 25 December 1974

Most *barrel tiles of clay or concrete* rely on screwed or nailed fixings located near the lagging edges only. This is unsatisfactory. There is the need to provide additional fastening in the leading half of the tile. This can be done as shown in OHPT-5 and OHPT-6.

1.8 Timber Structure - Guidance on the Use of Hurricane Straps

This procedure covers the use of hurricane straps for many common situations. The steps to be taken are:

- Identify the need for connectors.
- Select suitable connectors from manufacturers' catalogues.
- Determine the basic wind speed for the area, the design wind speed and the design wind pressure from Section 3.
- Determine design wind load from Section 4.
- Determine the number of connectors (or the frequency of connectors) required to resist the wind load using the appropriate part of Section 5.
- Adjust the spacing of timber members and/or connectors to achieve a regular and convenient set of construction details.

2 General Introduction to Hurricane Straps

The range of hurricane straps reviewed here include:

Rafter-to-purlin Clips As the name implies, these are used to connect purlins (or battens) to the main supporting rafters in roof construction. They may be used either singly or doubly at each rafter/purlin crossover.

These may also be used to connect the ends of rafters to timber wall plates. Between one and four clips may be used per junction, depending on the uplift force. (See OHPT-7.)

Rafter Connectors These may be used to connect the ends of rafters to sides of ridge boards. When bent at angles other than right angles they may be used to connect the ends of orthogonal rafters to hip rafters in hipped-roof construction. (See OHPT-8.)

Truss Anchors These are used where roof trusses or main rafters are supported by concrete belt beams (also known as ring beams). (See OHPT-9.)

Multi-purpose Straps As the name suggests, these items have many uses. They may be used to hold down lightweight buildings onto their foundations, to connect timber stud walls to sill plates, to connect upper storeys to lower storeys in timber construction and to supplement dedicated roof-member connections. (See OHPT-10.)

Mending Plates These are used at splice junctions. They are not very efficient in transferring direct loads and bending moments. Their appropriate use is as an economical connector where the loads or moments to be transferred are much less than the potential capacities of the connected members. (See OHPT-11.)

Moment Connectors These are used in pairs at butt junctions in beams where it is necessary to transfer significant bending moments. (See OHPT-12.)

Hurricane straps are supplied in several materials. The materials and thicknesses reviewed here are as follows:

- Galvalume coated steel plate with coating to ASTM A792M (AZ180). Structural quality base metal to ASTM A446M-93, grade D with a minimum tensile strength of 450 MPa (65,300 lbf/in^2) in thickness 1.00 mm (0.0394 in). Minimum yield of 345 MPa with elongation of 12%.

- Stainless steel plate to ASTM A240-S31603 with a minimum tensile strength of 485 MPa (70,300 lbf/in^2) in thickness 1.00 mm (0.0394 in). Minimum yield of 170 MPa with elongation of 40%.

The testing of the hurricane straps (see Appendix A) was carried out using two gauges of galvanised nails - 3.75 mm (0.148 in) and 3.15 mm (0.124 in). The length of the smaller nail was 35 mm (1.378 in) and the minimum length of the larger nail was 40 mm (1.575 in).

The tests used F8 seasoned slash pine (P. elliottii) with a nominal density of 625 kg/m^3 (39 lb/ft^3) at 12% moisture content and a mean modulus of elasticity of 12,000 N/mm^2 (1.74x10^6 lbf/in^2).

3 Wind Speed, Pressure and Load

3.1 Basic Wind Speed Table of the Caribbean (V)

This information is based mainly on the research and analysis undertaken by H C Shellard in 1970 and revised by B A Rocheford in 1981. Both of these meteorologists were attached to the Caribbean Meteorological Institute[4].

[4] Now named the Caribbean Institute of Hydrology and Meteorology

The wind speeds are 3-second gusts at 10 metres above ground in an open situation and likely to be exceeded not more than once in 50 years.

Place	**metres per second**	**miles per hour**	**Notes**
Guyana	22	49	
Trinidad	45	101	
Tobago, Grenada	50	112	
Grenadines, St Vincent, Barbados, St Lucia	58	130	Based on studies for Barbados only
Dominica, Montserrat	61	136	Interpolated
Antigua, Barbuda, St Kitts, Nevis, Anguilla, BVI, Turks & Caicos	64	143	Based on studies for Antigua only
Turks & Caicos, Bahamas, Jamaica, Cayman Islands, Northern Belize	58	130	Shellard-Rocheford-Davenport[5]
Southern Belize	50	112	Shellard-Rocheford-Davenport

Basic Wind Speeds (V) of the Caribbean
Table 1

3.2 Factors for Topography (S_1)

Recent hurricane events in the Caribbean have demonstrated the significant influence of topography on the levels of damage caused by the wind. Although most modern wind-load standards provide guidance and procedures for addressing this issue, the adjustment factors given in these documents do not fairly represent the range of effects experienced in the dramatic topographies of many Caribbean islands.

[5] The University of Western Ontario (Prof Alan Davenport) undertook a study of the hurricane hazard in 1985. This information has been used directly in CUBiC. It is not exactly comparable with the Shellard-Rocheford studies. However, the Davenport information has been used comparatively in conjunction with the Shellard-Rocheford studies to derive these figures.

Research work is ongoing in this area and it is expected that much better guidance will be available in the near future. In the meanwhile the factors in the following table may be used, although they do not represent the maximum range likely to be experienced in practice.

Topographic Condition	Topography Factor
Very exposed hill slopes and crests	1.1
Valleys shaped to produce a funnelling of the wind	1.1
Steep-sided enclosed depressions	0.9
All other situations	1.0

Topography Factor
Table 2

3.3 Factors for Ground Roughness and Height (S_2)

Since hurricane straps are used typically for connecting individual building components, only Class A structures are considered in the Table below. Class A structures include all units of cladding, roofing and their immediate fixings.

Height in metres (and storeys)	Open Country Coastlines	Suburbs Wooded Areas	Town Centres
4 (1 storey)	0.86	0.76	0.67
8 (2 storeys)	0.95	0.92	0.75
12 (3 storeys)	1.01	0.96	0.82

Ground Roughness and Height (S_2) Factor
Table 3

3.4 Design Wind Speeds (V_s)

$V_s = V \times S_1 \times S_2$

3.5 Wind Pressures (q)

Design Wind Speed (metres per second)	**Design Wind Pressure (N/m^2)**	**Design Wind Speed (miles per hour)**	**Design Wind Pressure (lbf/ft^2)**
20	245	45	5
25	383	56	8
30	552	67	12
35	751	78	16
40	981	89	20
45	1240	101	26
50	1530	112	32
55	1850	123	39
60	2210	134	46
65	2590	145	54
70	3000	157	63

Speed-to-pressure Conversions
Table 4
(see also Appendix C)

4 Wind Pressure Coefficients for Different Roof Shapes and Slopes

4.1 Hipped Roofs

This is the most favourable shape for a roof from the point of view of resisting wind forces. Although the theoretical maximum pressures (and suction forces) are not very different from those for gable roofs, the practical performance of hipped roofs in hurricanes is demonstrably superior to that of gable roofs. The reasons are:

- slightly lower maximum values of pressures and suction forces;
- more even distribution of pressures and suction forces over the roof surface;
- more favourable structural form leading to better (and less onerous) distribution of the loads from the roof to the walls;

- the practical need for better quality of workmanship for fabricating and erecting hipped roofs.

The table below gives external pressure coefficients (C_{pe}). In most cases it would be necessary to add an upward internal pressure coefficient of 0.2 to these figures.

Roof angle in degrees	**Maximum uplift pressure coefficient on interior of roof**	**Maximum uplift pressure coefficient on hip ridges**	**Maximum uplift pressure coefficient at horizontal ridge**
5	0.64	0.89	0.61
15	0.64	1.43	1.31
30	0.78	1.40	0.78
45	0.88	1.35	0.88

Pressure Coefficients for Hipped Roofs
Table 5

For edge areas where the higher forces apply, see OHPT-13.

Note: The hatched areas in OHPT-13 extend approximately 15% of the relevant dimensions of the sections of the roof.

4.2 Gable Roofs

The table below gives external pressure coefficients (C_{pe}). In most cases it would be necessary to add an upward internal pressure coefficient of 0.2 to these figures.

Roof angle in degrees	Maximum uplift pressure coefficient on interior of roof	Maximum uplift pressure coefficient on eaves and gable edges	Maximum uplift pressure coefficient at ridges
5	0.9	1.4	1.0
10	1.2	1.4	1.2
20	0.7	1.0	1.2
30	0.7	0.8	1.1
45	0.7*	0.7	1.1

Pressure Coefficients for Gable Roofs
Table 6

* There is also the possibility of a downward pressure coefficient of 0.3 in this case.

Unit wind load = Design wind pressure x pressure coefficient
= q x C_p

For edge areas where the higher forces apply, see OHPT-13.

Note: The hatched areas in OHPT-13 extend approximately 15% of the relevant dimensions of the sections of the roof.

4.3 Mono-pitch Roofs

For these roofs the maximum external pressure coefficient (C_{pe}) on the interior of the roof is 1.0 and at the edges 2.0. In most cases it would be necessary to add an upward internal pressure coefficient of 0.2 to these figures.

Unit wind load = Design wind pressure x pressure coefficient
= q x C_p

4.4 Eaves Overhangs

For these areas an upward pressure coefficient of 0.7 should be added to the appropriate external pressure coefficient (C_{pe}).

Unit wind load = Design wind pressure x pressure coefficient
= q x C_p

4.5 Balcony Roofs

Where the roof is at the same level as the main roof of the building, use an overall pressure coefficient (C_p) of 1.8 upward.

Where the building extends a storey or more above the balcony roof, use an overall pressure coefficient (C_p) of 0.9 upward.

For all cases, use an overall pressure coefficient (C_p) of 3.0 upward for the edges of the roof.

Unit wind load = Design wind pressure x pressure coefficient
= q x C_p

5 Allowable Loads

5.1 Rafter Purlin Clip (See OHPT-7.)

The permissible design load (unfactored) is 3 kN (670 lbf) using 3.75 mm nails and 2.8 kN (630 lbf) using 3.15 mm nails.

Notes:

1. It is important to use 4 nails per leg and to place nails in the holes closest to the inside corner of the clip.

2. Although both the stainless steel and the galvanised steel clips satisfy the requirements for the stated permissible design loads, the stainless steel clips have greater reserve capacity (about 10%).

Prescriptive Diagrams and Tables (see following sub-section)

5.2 Rafter Connector (See OHPT-8.)

The permissible design load (unfactored) is 3 kN (670 lbf) using 3.75 mm nails and 2.4 kN (540 lbf) using 3.15 mm nails.

Note:

1. Both the stainless steel and the galvanised steel clips satisfy the requirements for the stated permissible design loads.

5.3 Truss Anchor (See OHPT-9.)

The permissible design load (unfactored) is 8.2 kN (1840 lbf) using 3.75 mm nails with the 0.87 mm stainless steel and 2.0 mm galvanised steel anchors. The permissible design load (unfactored) is 7.8 kN (1750 lbf) using 3.15 mm nails with the 1.0 mm galvanised steel anchors.

Notes:

1 Although both the stainless steel and the galvanised steel clips satisfy the requirements for the stated permissible design loads, the 2.0 mm galvanised steel clips have greater reserve capacity (over 20% generally). However caution should be exercised in relying on this extra capacity since the test results showed significant variation.

2 BRC normally supplies the truss anchor in 1.0 mm galvanised steel. For 1.0 mm galvanised steel in conjunction with 3.75 mm nails the designer may use a permissible design load (unfactored) of 8.0 kN (1795 lbf).

5.4 Multi-purpose Strap (See OHPT-10.)

The permissible design load (unfactored) is 2.6 kN (584 lbf) using 3.75 mm nails.

5.5 Mending Plate (See OHPT-11.)

The permissible design moment (unfactored) is 0.88 kNm (7800 lbf-in) using 3.75 mm nails.

Note:

1 This should only be used for nominal connections because the connection has much less capacity than the main members.

5.6 Moment Connector (See OHPT-12.)

The permissible design moment (unfactored) is 2.5 kNm (22,000 lbf-in) for a 140 mm (5½ in) deep beam.

Note:

1 With this connector, the deeper the beam the greater the permissible design moment.

APPENDICES

A History of the CSTS Testing Programme

Damage to lightweight roofs is a common feature when hurricanes strike. Loss of roof sheeting is one form of failure. Loss of roof structure is the other form of failure. On Christmas Day in 1974 Cyclone Tracy devastated the city of Darwin in Northern Australia. In particular, the 24-gauge, arc-tangent profile, galvanised roof sheeting commonly used in that city suffered almost 100% losses. Because of that dramatic event, an exhaustive programme of testing was initiated and carried out in Australia. Coming out of that initial programme was the establishment of the Cyclone Structural Testing Station (CSTS) in Queensland, Australia. The CSTS is now regarded as one of the foremost research centres for hurricane-resistant structural components in the world.

In the 1980s the IBRD/IDB Phase II Schools project was in progress in Barbados. The structural engineer for Government's project office, Dr Prevala Sivaprakasapillai of Sri Lanka, encouraged BRC West Indies Limited to arrange for a series of tests at CSTS on roof sheeting fasteners. That initial programme was completed in 1988. A further series of tests was conducted in 1993-4 by CSTS for **BRC**. Loads for this programme were provided by Consulting Engineers Partnership Ltd (CEP). In the meanwhile **BRC** had also consulted with the UWI[6] Civil Engineering Department with a view to getting that institution involved in a similar testing programme.

Other initiatives in hurricane-resistant construction which came out of **BRC** included hurricane shutters in 1994.

BRC held preliminary discussions with CEP on hurricane straps starting in June 1994. Negotiations between **BRC** and CSTS started from December 1995. CEP was appointed to advise **BRC** on the testing programme in January 1996.

Throughout the testing programme there was interaction between CSTS, **BRC** and CEP in order to resolve problems with materials (timber and nails), loads, adjustment of connectors and hole locations.

Of particular interest is the cyclic testing procedures used in this programme. The research carried out after Cyclone Tracy demonstrated clearly that traditional static load tests do not give reliable information on the performance characteristics of lightweight building components in hurricanes. Regrettably, the vast majority of tests worldwide still use static procedures rather than cyclic-dynamic loading.

[6] The University of the West Indies, St Augustine, Trinidad

The Cyclone Structural Testing Station completed its testing, and its reporting thereon, in August 1997. The Report TS496 "Cyclic Loading of **BRC** Timber Connectors" is available for examination, on request, at **BRC** West Indies Limited.

B Guidance for Situations Not Covered in the Main Sections of the Document

Of necessity this document cannot cover the full range of situations likely to be met in practice. It is hoped that it can be seen to cover the use of the described hurricane straps in the more common applications.

For complex situations such as:

- unusual or dramatic topography *eg* cliff edges
- unusual ground roughness *eg* middle of cities
- temporary structures or, on the other hand, critically important structures
- buildings or structures of irregular shape
- very flexible structures

the standard BNS CP28 "Code of Practice for Structural Design"[7] provides comprehensive guidance on most situations likely to be met in practice. The appendices in BNS CP28 contain valuable background material to assist the interested practitioner in understanding the specific provisions in that standard.

C Wind Pressures from Wind Speeds

For values of wind speed between the values given in Table 4, the graph in OHPT-14 may be used facilitate interpolations.

[7]This is an official document of the Barbados National Standards Institution (BNSI). It is not currently on sale by the BNSI but photocopies are available from Mr Tony Gibbs, Consulting Engineers Partnership Ltd, tel (246) 426 5930, fax (246) 426 5935, Email concept@caribsurf.com.

B3.2 Detailing For Earthquakes

by Anthony C T Farrell, MSc, FICE, FIStructE
November 2000

1.0 Introduction

This paper deals with some of the detailing requirements of structures for seismic performance. The detailing of members and joints goes hand in hand with the philosophy of the derivation of design loads.

It is recognized and expected that even a building properly designed and constructed in accordance with minimum requirements of modern day codes can suffer significant damage in a large earthquake. It is however reasonable to expect that they will not collapse. "…The protection of life is reasonably provided but not with complete assurance… "

These words are a quotation from the commentary of the 1996 edition of the Structural Engineers Association of California (SEAOC) Blue Book. All codes recognize that there are no 100% guarantees in seismic design. There are no earthquake proof buildings.

2.0 Some Principles

Diagram 2 illustrates the kind of behaviour for which code writers aim. Code writers and legislators have so far taken the view that it is not economically feasible to design buildings generally for an elastic response to the largest earthquake and thus engineers aim to follow the V_{SMRF} line which implies yielding and plastic rotation of hinges with energy absorption as the structure deflects.

Thus design forces are generally a fraction of the strength required to respond in an elastic manner to large earthquakes.

Carrying this through in a consistent manner then the structure must respond in a "ductile" manner.

This paper intends to deal both with reinforced concrete and structural steel and two other aims of structural detailing have to be mentioned.

That is the strong column – weak beam principle. Simply put, the columns must be stronger than the beams attached to the same joint to avoid the formation of hinges in the columns. Diagram 1 illustrates the condition to be avoided and why it is to be avoided.

Codes are also now recognizing the importance of redundancy to the behaviour of a structure. Again in simple terms the greater the number of alternative load paths in a structure the better are its chances of surviving a severe seismic event.

In the UBC 1997 recommendations, redundancy factors should lie between 1.0 and 1.5. Less redundant systems are assigned higher values and vice versa.

3.0 Detailing

This paper intends to give an introduction to details required both for reinforced concrete and structural steel. Details are mainly but not totally restricted to Special Moment Resisting Frames.

4.0 Reinforced Concrete

Requirements for reinforced concrete are given here for flexural members, members carrying axial loads plus flexure and concrete shear walls and are taken from Chapter 21 of ACI-318 Building Code Requirements for Structural Concrete.

These cover the requirements for the following:

- Materials
- Seismic hoops
- Dimensional limitations
- Flexural members
- Members subject to Axial load and bending
- Shear walls

Almost all of the illustrations (Figures 7 to 44) have been copied from a presentation on detailing by SK Ghosh in March 1997.

5.0 Structural Steel

Structural steel in theory is an excellent material for absorbing energy since the material itself exhibits a degree of ductile behaviour. The problem lies in the connections.

Until the 1994 Northridge earthquake American engineers utilized welded special moment resisting frames (SMRF) as one of their most fashionable and accepted means of structural framing. These frames cater for gravity as well as the effects of seismic actions and wind storm without the introduction of walls and braces which Architects dislike.

However after the Northridge earthquake particularly the US engineering fraternity was surprised and dismayed at the amount of unexpected brittle fractures that occurred to the connections of welded frames that until then, had been thought to be properly designed. The alarming aspect was that the fractures apparently were initiated at very low levels of plastic demand, and in some cases, while the structure remained essentially elastic. These failures could lead to potential development of local collapse mechanisms.

Because of these concerns the US Federal Emergency Management Agency (FEMA) funded (at enormous cost) a major 6-year programme that, as of about September 2000, was considered basically complete.

The programme's aim was to develop reliable, practical and cost effective guidelines for the design and construction of new steel moment-resisting frames as well as for the upgrade of existing ones. The research led to the publication of four (4) books of recommendations (FEMA–350 to 353-2000) and six (6) State-of-the-Art Reports (FEMA-355 A to F – 2000). This documentation was first released officially at a 2-day seminar workshop on 11th September 2000.

These recommendations have not yet been codified but it is a safe bet that it will not be long before they are.

Interestingly enough the research and recommendations covered extended end plate connections which are used in the Caribbean far more than site welded connections.

The FEMA books are listed in an Appendix to this paper.

6.0 Pre-qualified Connections

Subsequent to Northridge and prior to the issue of these documents it had been recommended that <u>all</u> connections used in moment resisting frames for seismic resistance be qualified for adequacy through a programme of prototype testing. This recommendation was adopted by FEMA-302, the1997 AISC Seismic Provisions and the UBC 1997.

The connections listed in these FEMA publications can now be considered to be pre-qualified within the limitations of the testing carried out. Selection from a list of pre-qualified connections is clearly easier than expensive testing of project-specific connections.

The methodology used by all these assemblies is to force the hinge location away from the column face. The basic pre-qualified connections are as shown in Fig. ……….. It is recommended that the reader consult "FEMA 350: Recommended Seismic Design Criteria for New Steel Moment Frame Buildings".

The fundamental requirements are:

- The determination of the location of plastic hinges
- The determination of the probable Plastic Moments at Hinges
- The determination of the shear in the Plastic Hinge
- The determination of strength or strength demands at critical locations

APPENDIX 1

List of FEMA Books

- **Recommended Seismic Design Criteria for New Steel Moment Frame Buildings.** This publication provides recommended criteria for the design of new steel moment frame buildings to resist the effects of earthquakes (FEMA-350, 2000)

- **Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment Frame Buildings.** This publication provides recommended methods to evaluate the probable performance of existing steel moment-frame buildings in future earthquakes and to retrofit these buildings for improved performance (FEMA-351, 2000)

- **Recommended Post-earthquake Evaluation and Repair Criteria for Welded, Steel Moment Frame Buildings.** This publication provides recommendations for performing post-earthquake inspections to detect damage in steel moment-frame buildings following an earthquake, evaluating the damaged buildings to determine their safety in the post-earthquake environment, and repairing damaged buildings (FEMA-352, 2000)

- **Recommended Specifications and Quality Assurance Guidelines for Steel Moment Frame Construction for Seismic Applications.** This publication provides recommended supplemental specifications and recommended procedures to ensure that steel moment frames are constructed with sufficient construction quality to perform as intended when subjected to severe earthquake loading (FEMA-353, 2000)

Detailed derivations and explanations of the basis for these design and evaluation recommendations may be found in a series of State of the Art Reports. These reports include:

- **State of the Art Report on Base Metals and Fracture.** This report summarizes current knowledge of the properties of structural steels commonly employed in building construction, and the production and service factors that affect these properties (FEMA-355A, 2000).

- **State of the Art Report on Welding and Inspection.** This report summarizes current knowledge of the properties of structural welding commonly employed in building construction, the effect of various welding parameters on these properties, and the effectiveness of various inspection methodologies in characterizing the quality of welded construction (FEMA-355B, 2000).

- **State of the Art Report on Systems Performance.** This report summarizes an extensive series of analytical investigations into the demands induced in steel moment-frame buildings designed to various criteria, when subjected to a range of different ground motions. The behaviour of frames constructed with fully restrained, partially restrained and fracture-vulnerable connections is explored for a series of ground motions, including motion anticipated at near-fault and soft-soil sites (FEMA-355C, 2000).

- **State of the Art Report on Connection Performance.** This report summarizes the current state-of-knowledge of the performance of different types of moment-resisting connections under large, inelastic deformation demands. It includes information on fully restrained, partially restrained, and partial strength connections, both welded and bolted, based on laboratory and analytical investigations (FEMA-355D, 2000).

- **State of the Art Report on Past Performance of Steel Moment-Frame Buildings in Earthquakes.** This report summarizes investigations of the performance of steel moment-frame buildings in past earthquakes, including the 1995 Kobe, 1994 Northridge, 1992 Landers, 1992 Big Bear, 1989 Loma Prieta and 1971 San Fernando events (FEMA 355E, 2000).

- **State of the Art Report on Performance Prediction and Evaluation.** This report describes the results of investigations into the ability of various analytical techniques, commonly used in design, to predict the performance of steel moment-frame buildings subjected to earthquake ground motion. Also presented is the basis for performance-based evaluation procedures contained in the design criteria and guideline documents, *FEMA-350 to FEMA-353.* (FEMA-355F, 2000).

In addition to the recommended design criteria and the State of the Art Reports, a companion document has been prepared for building owners, local community officials and other non-technical audiences who need to understand this issue. *A Policy Guide to Steel Moment-Frame Construction* (FEMA 354, 2000), addresses the social, economic, and political issues related to the earthquake performance of steel moment-frame buildings. *FEMA 354* also includes discussion of the relative costs and benefits of implementing the recommended criteria.