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**TNO-rapport**

**96-CON-R0781-3**

# Hurricane Hazard at the Netherlands Antilles and wind loads on buildings

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# 1 INTRODUCTION

At the request of the Netherlands Red Cross department Bonaire, Netherlands Antilles, TNO has carried out an inventory study on design wind loads on buildings due to hurricanes and into the provisions to be taken for buildings and structures to withstand these wind loads. This report deals with the wind load phenomena associated with hurricanes, with the calculation of wind loads and the design and remedial measures that may be applied to buildings and structures, both to be designed and existing ones. The study is not intended to give final and definitive answers to the question of recommended design loads; rather, this report formulates some important issues, such as the required structural safety levels of buildings, and gives recommendations on principal choices necessary to be made.

## 2 HURRICANES AND OTHER EXTREME WIND EVENTS

*Tropical cyclones*
The terms hurricane and typhoon are regionally specific names for a strong tropical cyclone. A tropical cyclone is the generic term for a non-frontal synoptic scale low-pressure system over tropical or sub-tropical waters with organised convection and definite cyclonic surface wind circulation

Tropical cyclones with maximum sustained surface winds of less than 17 m/s are called *tropical depressions*. Once the tropical cyclone reaches winds of at least 17 m/s they are typically called a *tropical storm* and assigned a name. If winds reach 33 m/s, then they are called: a *hurricane* in the North Atlantic Ocean, the Northeast Pacific Ocean east of the dateline, (or the South Pacific Ocean east of 160E); a "typhoon" (the Northwest Pacific Ocean west of the dateline); a "severe tropical cyclone" (the Southwest Pacific Ocean west of 160E or Southeast Indian Ocean east of 90E); a "severe cyclonic storm" (the North Indian Ocean); and a "tropical cyclone" (the Southwest Indian Ocean).

Note that the definition of "maximum sustained surface winds" depends upon who is taking the measurements. The World Meteorology Organisation guidelines suggest using a 10 min average to get a sustained measurement. Most countries use this as the standard. However, the National Hurricane Center (NHC) and the Joint Typhoon Warning Center (JTWC) of the USA use a 1 minute averaging period to get sustained winds. This difference may provide complications in comparing the statistics from one weather station to another as using a smaller averaging period may slightly raise the number of occurrences. According to (Simiu & Scanlan, 1986) the difference may amount to 25 % in the calculated extreme wind speed.

*Mid-latitude storms*
The tropical cyclone is a low-pressure system which derives its energy primarily from evaporation from the sea in the presence of high winds and lowered surface pressure and the associated condensation in convective clouds concentrated near its center. Mid-latitude storms, such as low pressure systems with associated cold fronts, warm fronts, and occluded fronts, primarily get their energy from the horizontal temperature gradients that exist in the atmosphere.

*Tornadoes*
While both tropical cyclones and tornadoes are atmospheric vortices, they have little in common. Tornadoes have diameters on the scale of hundreds of meters and are produced from a single convective storm (i.e. a thunderstorm or cumulonimbus). A tropical cyclone, however, has a diameter on the scale of thousands of kilometres and is comprised of several to dozens of convective storms. Additionally, while tornadoes require substantial vertical shear of the horizontal winds (i.e. change of wind speed and/or direction with height) to provide ideal conditions for tornado genesis, tropical cyclones require very low values (less than 10 m/s) of tropospheric vertical shear in order to form and grow. These vertical shear values are indicative of the horizontal temperature fields for each phenomenon: tornadoes are produced in regions of large temperature gradient, while tropical cyclones are generated in regions of near zero horizontal temperature gradient. Tornadoes are primarily an over-land phenomenon, as solar heating of the land surface usually contributes toward the development of the thunderstorm that spawns the vortex. In contrast, tropical cyclones are purely an oceanic

phenomenon: they die out over land due to a loss of a moisture source. Lastly, tropical cyclones have a lifetime that is measured in days, while tornadoes typically last on the scale of minutes.

*Sub-tropical cyclones*

A sub-tropical cyclone is a low-pressure system existing in the tropical or subtropical that has characteristics of both tropical cyclones and mid-latitude cyclones. Therefore, many of these cyclones exist in a weak to moderate horizontal temperature gradient region, but also receive much of their energy from convective clouds, like tropical cyclones. Often, these storms have a radius of maximum winds which is farther out, on the order of 100-200 km from the center, than what is observed for purely tropical systems. Additionally, the maximum sustained winds for sub-tropical cyclones have not been observed to be stronger than about 33 m/s.

*Classification of Atlantic hurricanes*

The USA utilises the Saffir-Simpson hurricane intensity scale for the Atlantic and Northeast Pacific basins to give an estimate of the potential flooding and damage to property given a hurricane's estimated intensity:

| Saffir-Simpson Category | Maximum sustained wind speed (m/s,kt) | | Minimum surface pressure (mb) | Storm surge (m) |
|---|---|---|---|---|
| 1 | 33-42 m/s | [64-83 kt] | >= 980mb | 1.0-1.7 m |
| 2 | 43-49 | [84-96] | 979-965 | 1.8-2.6 |
| 3 | 50-58 | [97-113] | 964-945 | 2.7-3.8 |
| 4 | 59-69 | [114-135] | 944-920 | 3.9-5.6 |
| 5 | > 69 | [> 135] | < 920 | > 5.6 |

1: MINIMAL: Damage primarily to shrubbery, trees, foliage, and unanchored homes. No real damage to other structures. Some damage to poorly constructed signs. Low-lying coastal roads inundated, minor pier damage, some small craft in exposed anchorage torn from moorings. Example: Hurricane Jerry (1989)

2: MODERATE: Considerable damage to shrubbery and tree foliage; some trees blown down. Major damage to exposed mobile homes. Extensive damage to poorly constructed signs. Some damage to roofing materials of buildings; some window and door damage. No major damage to buildings. Coast roads and low-lying escape routes inland cut by rising water 2 to 4 hours before arrival of hurricane center. Considerable damage to piers. Marinas flooded. Small craft in unprotected anchorage's torn from moorings. Evacuation of some shoreline residences and low-lying areas required. Example: Hurricane Bob (1991)

3: EXTENSIVE: Foliage torn from trees; large trees blown down. Practically all poorly constructed signs blown down. Some damage to roofing materials of buildings; some wind and door damage. Some structural damage to small buildings. Mobile homes destroyed. Serious flooding at coast and many smaller structures near coast destroyed; larger structures near coast damaged by battering waves and floating debris. Low-lying escape routes inland cut by rising water 3 to 5 hours before hurricane center arrives. Flat terrain 1.5 m of less above sea level flooded inland 12 km or more. Evacuation of low-lying residences within several blocks of shoreline possibly required. Example: Hurricane Gloria (1985)

4: EXTREME: Shrubs and trees blown down; all signs down. Extensive damage to roofing materials, windows and doors. Complete failures of roofs on many small residences. Complete destruction of mobile homes. Flat terrain 3 m of less above sea level flooded inland as far as 16 km. Major damage to lower floors of structures near shore due to flooding and battering by waves and floating debris. Low-lying escape routes inland cut by rising water 3 to 5 hours before hurricane center arrives. Major erosion of beaches. Massive evacuation of all residences within 500 m of shore possibly required, and of single-story residences within 3 km of shore. Example: Hurricane Andrew (1992)

5: CATASTROPHIC: Shrubs and trees blown down; considerable damage to roofs of buildings; all signs down. Very severe and extensive damage to windows and doors. Complete failure of roofs on many residences and industrial buildings. Extensive shattering of glass in windows and doors. Some complete building failures. Small buildings overturned or blown away. Complete destruction of mobile homes. Major damage to lower floors of all structures less than 4.5 m above sea level within 500 yards of shore. Low-lying escape routes inland cut by rising water 3 to 5 hours before hurricane center arrives. Massive evacuation of residential areas on low ground within 8 to 16 km of shore possibly required. Example: Hurricane Camille (1969)

# 3 HURRICANE HAZARD FOR THE NETHERLANDS ANTILLES

## *3.1 Extreme value analysis*

For the purpose of this study, data on hurricane experience in the past at the Netherlands Antilles has been supplied by the Meteorological Service of the Netherlands Antilles and Aruba (Meteorological Service, 1994). This data concerns a list of hurricanes with maximum sustained wind speeds and nearest distance to the leeward and windward islands of the Antilles. As a general observation the hurricane activity at the leeward islands is far lower than at the windward islands. For the leeward islands, the frequency is on average once in four years, for the windward islands approximately once every year. The data supplied by the Meteorological Services does not rely on measurements of the wind speeds on the windward and leeward islands itself, but merely reports the maximum sustained wind speeds that occurred in the passing hurricanes. Also, it is not unambiguous whether the historic data concern extreme gradient wind speeds or surface wind speeds. However, since there is no further information available, in this report these data are used.

Based on these historic data, an estimate of the probability of exceedence of certain wind speeds has been made by using some calculation models for hurricane wind speeds. As a first step, an estimate has been made of the wind speed at the minimal distance at which the hurricane eye has passed the islands. This estimate is based on an assumed mean radius of the eye of the hurricane of 40 km (Vickery & Twisdale, 1995a) and on a model of the attenuation of wind speeds with distance derived from (Simiu & Scanlan, 1986). Finally, for hurricanes that pass on the north or east-side of the islands, the translation velocity of the eye of the hurricane has been subtracted from the reported maximum sustained wind speeds.

After this processing of the historic data, the hurricane wind speeds have been ranked and this ranked data has been fitted to a Gumbel-probability distribution and to a Weibull distribution:

The cumulative Gumbel distribution is: $P(u < U) = \exp(-\exp(-\frac{(U - U_v)}{a_v}))$

The cumulative Weibull distribution is: $P(u < U) = 1 - \exp\left(-\left(\frac{U}{v_o}\right)^k\right)$

For winds on moderate latitudes (50 degrees North), the Gumbel distribution works well for extreme wind speeds. However for hurricanes it is suggested by (Georgiou, Davenport and Vickery, 1983) that a Weibull distribution is more appropriate. The results are presented in figures 3.1 through 3.4. In these figures, the maximum sustained wind speed (1 minute average) has been plotted against the probability of exceedence in a 100 year period. An optimal fit should yield a straight line.

Figure 3.1: Gumbel fit of data on hurricane wind speeds near the Leeward islands

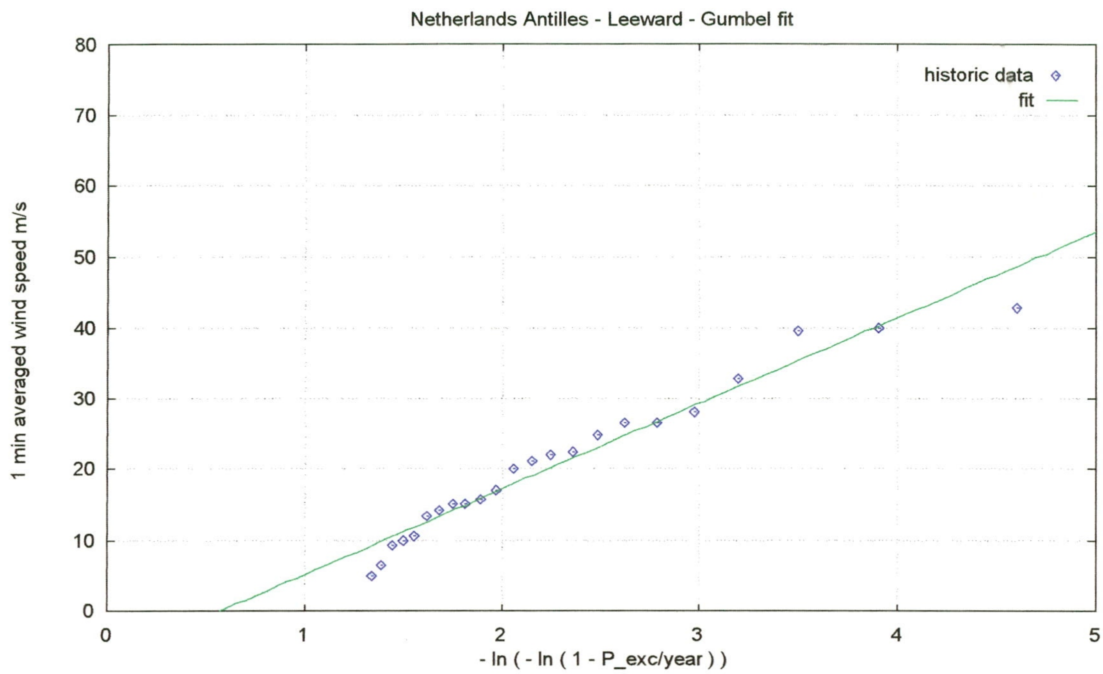



Figure 3.2: Gumbel fit of data on hurricane wind speeds near the Windward islands

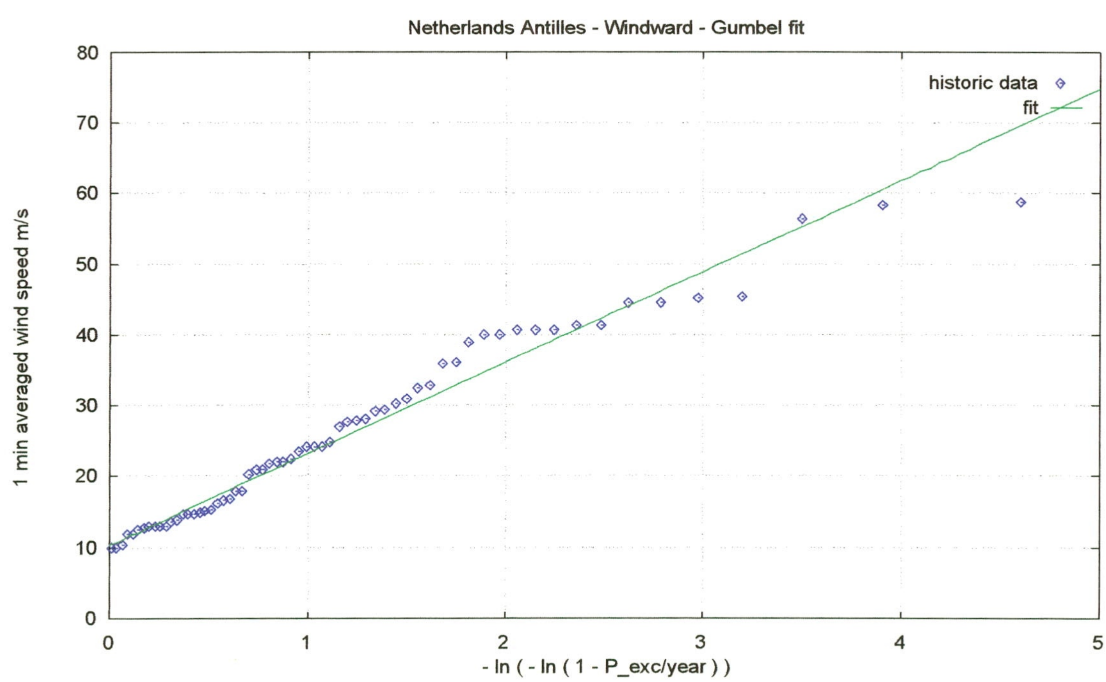


Figure 3.3: Weibull (2 parameter) fit of data on hurricane wind speeds near the Leeward islands

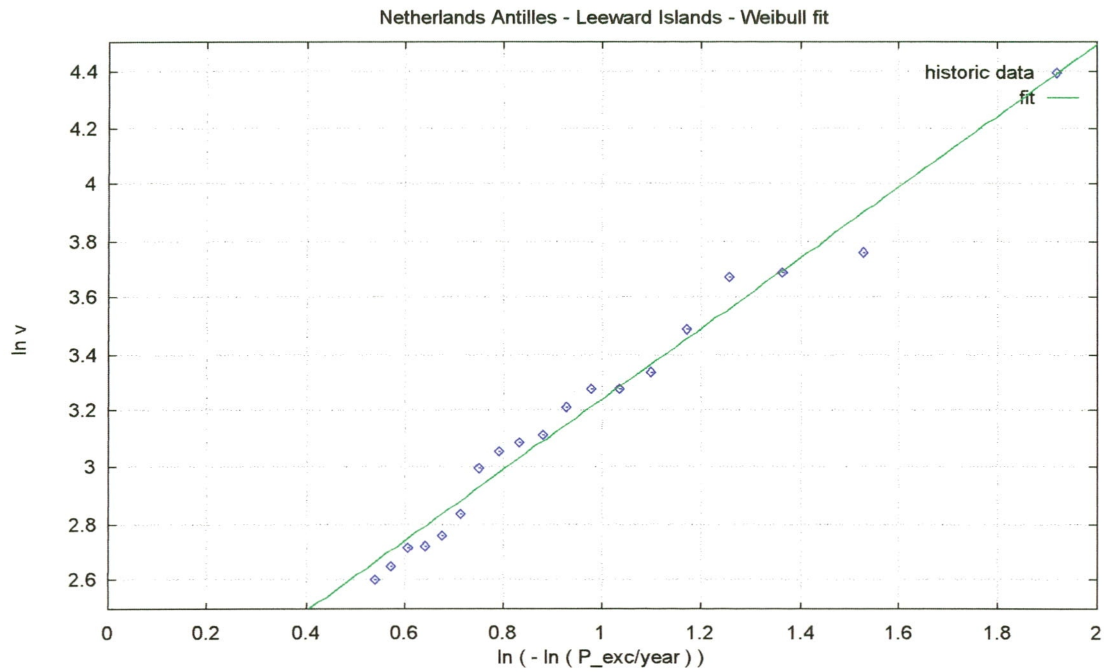
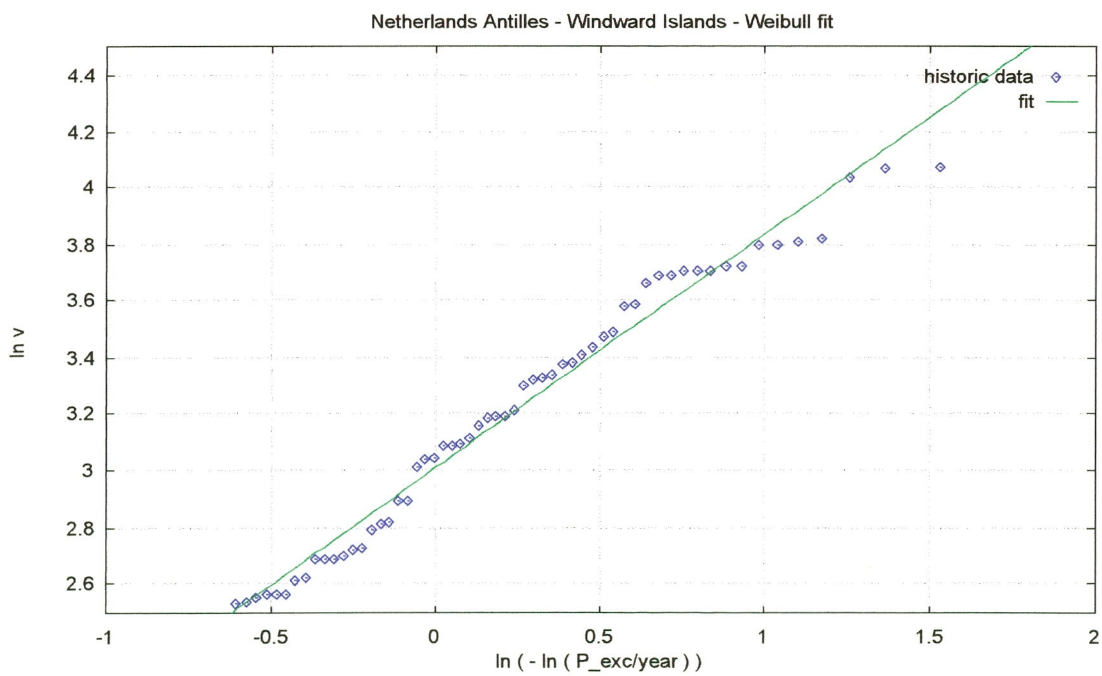



Figure 3.4: Weibull (2 parameter) fit of data on hurricane wind speeds near the Windward islands



As a rough estimate, it can be concluded from figures 3.1-3.4 that a Weibull distribution gives a better fit than a Gumbel distribution. For the 100-year extreme sustained wind speed (1 minute averages) the modus and the spread of the Gumbel distribution is given in Table 3.1. Modus and slope of the Weibull distribution for a 1-year extreme sustained wind speed are given in Table 3.2.

Table 3.1: Parameters of Gumbel-fit on 100-year extreme sustained wind speed data for a 100 year period.

| area | $U_v$ (m/s) | $a_v$ (m/s) |
|---|---|---|
| leeward | 48.7 | 12.1 |
| windward | 69.6 | 12.9 |

Table 3.2: Parameters of Weibull-fit on 1-year extreme sustained wind speed data for a 100 year period.

| area | $v_o$ (m/s) | k |
|---|---|---|
| leeward | 7.32 | 0.80 |
| windward | 20.3 | 1.21 |

The fit indicates that the spread (the uncertainty in the data) is relatively large for the Leeward islands, whereas the data for the Windward islands show a more consistent view. From both distributions, the maximum sustained wind speed (1 minute average) can be calculated, that will be exceeded once in 100 years; the results are given in Table 3.3 and 3.4. It appears that the difference between the Gumbel and Weibull fit is relatively small for the Windward islands and larger for the Leeward islands. Due to the large spread for the Leeward islands, extrapolation outside this 100 year period should be done with care.

Also the values applicable to shorter and longer return periods are given in Table 3.3 and 3.4. It should be noted that the data are a coarse estimate at this moment and should be further evaluated using the path statistics of hurricanes. Therefore the data given in this chapter is to be considered a first estimate.

Table 3.3: Estimated extreme wind speed (1 min. average) on the islands, with mean return period between 10 and 1500 year period, based on Gumbel fit, and comparison with values calculated for Amsterdam

| return period (years) | Leeward Islands | Windward Islands | Netherlands |
|---|---|---|---|
| 10 | 20.9 m/s | 39.9 m/s | 24.5 |
| 50 | 40.3 | 60.6 | 27.5 |
| 100 | 48.7 | 69.6 | 28.8 |
| 500 | 68.2 | 90.3 | 31.8 |
| 1000 | 76.6 | 99.3 | 33.0 |
| 1500 | 81.5 | 104.5 | 38.1 |

Table 3.4: Estimated extreme wind speed (1 min. average) on the islands, with mean return period between 10 and 1500 year period, based on Weibull fit

| return period (years) | Leeward Islands | Windward Islands |
|---|---|---|
| 10 | 20.8 m/s | 40.3 m/s |
| 50 | 40.5 | 62.4 |
| 100 | 49.7 | 71.4 |
| 500 | 72.4 | 91.4 |
| 1000 | 82.7 | 99.7 |
| 1500 | 88.8 | 104.5 |

From tables 3.3 and 3.4 it becomes clear that in case of a return period of 100 years, for the Leeward islands Saffir Simpson scale 2 may apply (mean sustained wind speeds 43-49 m/s), for the Windward islands scale 4 to 5 is appropriate (mean sustained wind speeds 59 - 69 m/s). Also it is obvious that the values for the larger return periods in Amsterdam are much smaller than the values at both Leeward and Windward islands.

The data have been compared to studies performed for locations at the east coast of the USA (Georgiou, Davenport and Vickery, 1983) and (Vickery and Twisdale, 1995a, 1995b). Figure 3.5 is taken from the first mentioned study and displays the wind speeds for various mean return periods. It should be noted that these data are given as hourly mean wind speeds. The ratio between 1 minute averaged wind speeds and hourly averaged wind speeds is approximately 1.25. For convenience, table 3.5 gives the extremes of the hourly mean wind speed for the Leeward and Windward islands.

Figure 3.5: USA Coastline variation of mean hourly wind surface speeds (Georgiou et al, 1983)

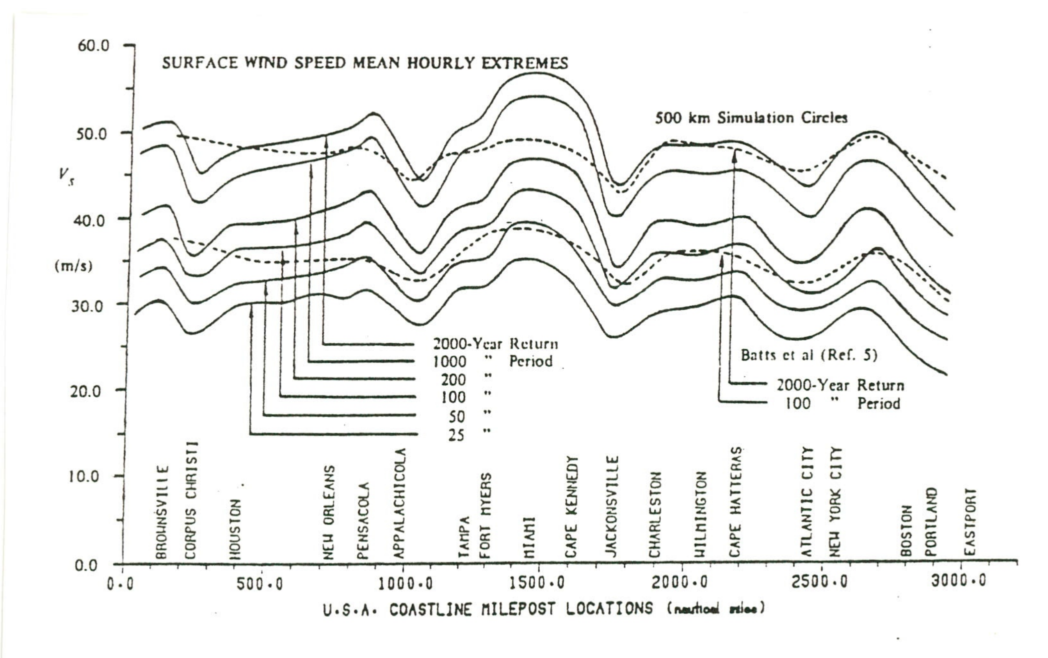


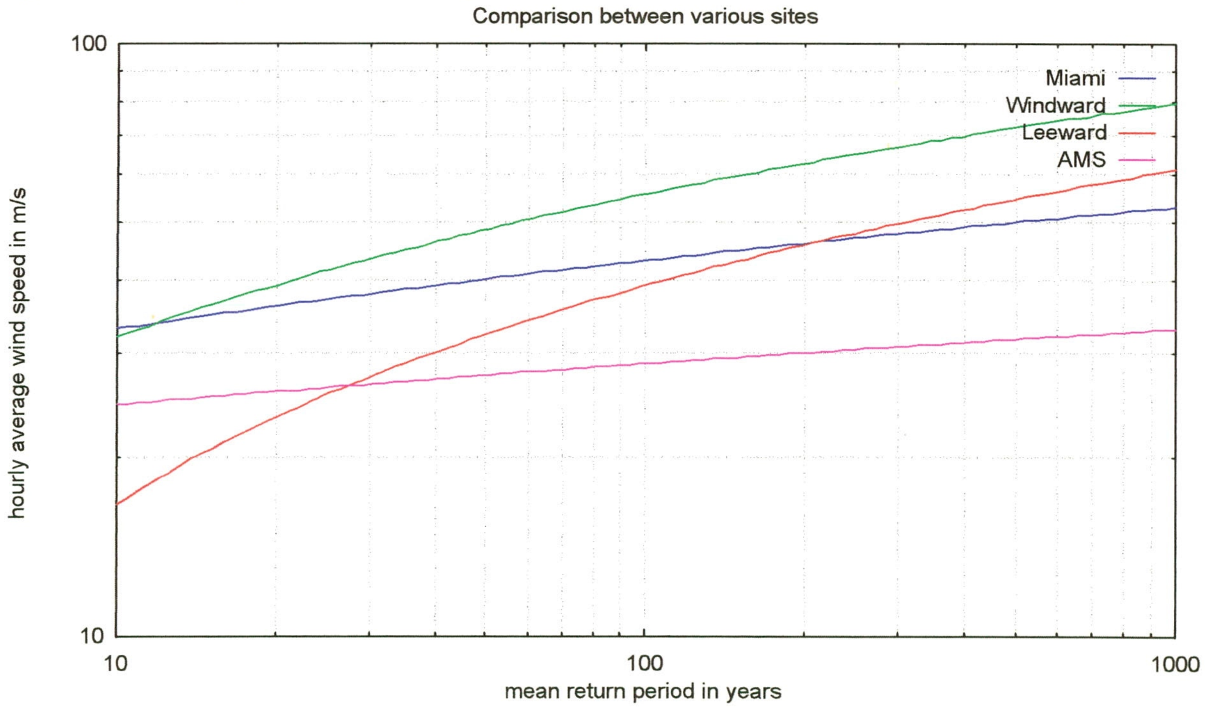
Table 3.5: Extreme hourly mean wind speed on the islands, with equivalent mean return period of 50 and 100 years, based on a Gumbel fit

| Return Period | Leeward Islands | Windward Islands |
|---|---|---|
| 50 years | 32.2 m/s | 48.5 m/s |
| 100 years | 40.0 m/s | 55.7 m/s |

The hurricane Luis, that passed the Windward Islands in September 1995 had a maximum wind speed at the airport of St. Maarten of approximately 70 knots (= 36 m/s). This wind storm has a mean return period of approximately 16 years according to the statistics derived from historical data. So this event is not at all to be considered rare at the Windward islands.

The estimated values for the Windward islands are higher than for instance the extreme wind speeds for the Miami area (extreme hourly mean wind speed of 43 m/s at 100 years return period). For comparison, in figure 3.6 the wind velocity for various return periods are given for the Leeward and Windward Islands, for Miami and for Amsterdam. The general level of the extreme wind speeds in the Caribbean is higher than for Western Europe (as would be expected), but also the extreme wind speeds increase at a higher rate for increasing return periods. It is remarked here that the extreme value analysis for the Netherlands Antilles and Aruba may be influenced by the results of future discussions on the nature of the historic hurricane wind speed data.

Figure 3.6: Probability of extreme wind speeds in the Caribbean and Amsterdam.



It is remarked here that no other references or literature have been traced containing extreme wind speed estimates in the area of the Leeward and Windward islands. It is strongly recommended to have such information available, to make a calculation, such as given above. Alternatively, one can consider to establish the probability functions based on theoretical analyses such as Monte Carlo simulations, as has been done in studies that have led to the USA buildings codes, see for example (Georgiou, Davenport, Vickery, 1983) and (Vickery and Twisdale, 1995a, 1995b). Especially the observation or assumption in literature, that the intensity of hurricanes may be bounded to the upper side, is important. This may imply that extrapolation to very large return periods are overly conservative.

### *3.2 Adoption of extreme wind speeds to design buildings and structures*

The wind load on a building is proportional to the squared wind velocity. The wind force F on a part of a building can be calculated from:

$$F = C_F \cdot A \frac{1}{2} \rho v^2$$

where:

| | |
|---|---|
| F | is the wind force, in N; |
| v | is the wind speed, in m/s; |
| ρ | is the density of air, 1.25 kg/m$^3$ |
| A | is the area of the part of the building, in m$^2$; |
| $C_F$ | is a wind shape factor, taking into account the shape of the part of the building and wind flow pattern around the part of the building. |

A very principal question is which return period for extreme wind events has to be the basis for the wind load used in design calculations of buildings and structures. The current practice in the Netherlands is based on a probability of exceedence of approximately 0.03 within a reference period of 50 years (NEN 6700, NEN 6702). This implies a mean return period of approximately 1400 years for a design storm.

The target set in newly developed European standards (ENV 1991-1) is even higher, at a mean return period of approximately 5000 years. Since this has too important implications for building economics, it has been decided in the Netherlands to use the mean return period of the design storm of 1400 years.

The procedure in the Netherlands building codes is that the design calculations are made on the basis of an extreme hourly mean wind speed having a mean return period of 50 years. This wind speed varies between 27.5 m/s and 30 m/s in the Western parts of the Netherlands. This wind speed is transferred into a design wind loading, representative for the maximum gust during an hour, which is described in section 3.4.

In addition, the wind loads are multiplied by a load factor of 1.5 to find the value, associated with the return period of 1400 years. Since the wind loads are proportional to the squared wind velocity, the use of a load factor of 1.5 means that the wind speed with a return period of 50 years is multiplied by $\sqrt{1.5}$

= 1.22 to define the wind speeds for the return period of 1400 years. This leads to hourly mean wind speeds of 34 to 36 m/s, in the western parts of the Netherlands.
The probability of exceedence of the design wind load is significantly reduced by application of the load factor.

The same procedure is also used in the US-code ANSI 58.1. However, from the studies that form the basis of this code it becomes clear that in the case of wind loads due to hurricanes, a load factor of 1.5 is not sufficient to increase the return period of the design storm to target levels of 1400 to 5000 years. For example, from figure 3.6 it is concluded that for the Florida area, the ratio in hourly mean wind speed for return periods of 50 years and 1500 years is approximately 1.4. The required load factor on the wind load would be $(1.4)^2 = 2$. Nevertheless, this consequence has not been taken into account in the US codes, and usually the calculations are carried out by applying the regular factor of safety of 1.5.

From the wind speed data of the Netherlands Antilles in table 3.4, it can be concluded that the ratio of the hourly mean wind speeds with a return period of 50 years and 1500 years is about 2 for the Leeward islands and about 1.7 for the Windward islands. The required load factor on the wind load then would be a factor of 4 for the Leeward Islands and a factor of 2.9 for the Windward Islands.

The conclusion to be drawn from table 3.5 is that if one considers to design on wind speeds having a 1500 year return period in a hurricane prone area as the Netherlands Antilles, one would have to use a *vastly higher* wind load than for a mid latitude country in Western Europe such as the Netherlands. The 1500 year return wind speed for the Windward islands is more than twice that of the value in the Netherlands, and the wind load will be more than five times the value in the Netherlands. Also for the Leeward Islands, the conclusion is that adopting the same target safety level as in the Netherlands would imply the application of far higher (at least three times) wind loads, than currently used. This may look surprisingly, as the extreme wind speeds for low return periods (for instance 10 years) are even lower than in the Netherlands.

This conclusion concerning the high wind velocities for return periods of 1500 years is formulated with the remark that the extrapolation of the historic hurricane wind speed data, as presented in this report, to very high return periods must be done with great care. As recommended in the previous section, further research in clarifying the background of the historic wind speed data and establishing the extreme wind speeds is necessary. Therefore, the above presented factors on the wind load are only indications.

### *3.3 Adoption of extreme wind loads for designing buildings and structures*

Given the very high wind speeds for return periods in the order of 1500 years, one should consider if the values still are an economically and socially optimal basis for the design of buildings and structures. Since wind load is proportional to the squared of the wind speed, the economic impact of the associated high wind loads for the design of buildings is enormous. The application of this design assumption without farther consideration may lead to a substantial increase in buildings costs and would cause a change in building practice.

As already indicated in the previous section, the 1500 year return period as established in the Netherlands is the result of an economic and safety optimisation, where costs of investment (design and building) have been compared to the cost of material and economic losses when buildings and structures fail in extreme conditions. This optimisation has been carried out prior to the revision of the Netherlands codes in 1990 (Vrouwenvelder, 1983). It is expected that such a economic and safety optimisation will produce a different optimal return period for extreme wind speed to be used in the design of buildings in the Netherlands Antilles and Aruba. It may lead to a lower optimal return period, but will inevitably lead to an accepted higher probability of failure of the building.

This means that the following principal choices need to be made:

- What is actually the structural safety level to be expected from buildings and structures?
- Which return periods of a extreme storm can be considered as an optimal balance between investment costs in building and the risk of loss (direct, economic and human) for the particular situation of the Netherlands Antilles.

To answer these questions, it is strongly recommended that a risk evaluation will be conducted. Without doubt, the need for a proper risk evaluation will apply to buildings with specific relevance to the society, such as shelters, hospitals and buildings related to public services, such as power stations, important transmission links and communication provisions.

Based on this risk evaluation, one may decide to introduce various levels in the required structural safety of a building. This classification ('safety classes') has also been introduced in the Netherlands codes in 1990 and may give a reason to make higher investments in the structural reliability of essential structures. Also, the use of preventive measures (strengthening, protecting of vulnerable buildings) may be taken into account in such safety classes.

According to ISO 2394, a reliability differentiation into safety classes must be based on the following four considerations:

- the cause and mode of failure implying that a structure or structural system which would be likely to collapse suddenly without warning should be designed for a higher degree of reliability than one for which a collapse is preceded by some kind of warning in such a way that measures can be taken to limit the consequences;
- the possible consequences of failure in terms of risk to life, injury, potential economic losses and the level of social inconvenience;
- the expense, level of effort and procedures necessary to reduce the risk;
- social and environmental conditions in a particular location.

ISO 2394 and ENV 1991-1 propose three safety classes, which may be defined as follows:

**Class 1** Risk to life, given a failure, is low and also economic and social consequences are small or negligible. Examples are agricultural structures, one family houses, lighting poles, traffic signs.

**Class 2** Risk to life, given a failure, is medium and economic and social consequences are considerable. Examples are flats, offices, industrial buildings.

**Class 3** Risk to life, given a failure, is high, or economic or social consequences are very large. Examples are main bridges, hospitals, shelters, high rise buildings, power stations, command and control centres.

The internationally accepted target design wind loads in highly developed areas may have return periods in the order of magnitude as mentioned below:

| | |
|---|---|
| Class 1 | 1000 years |
| Class 2 | 2500 years |
| Class 3 | 5000 years |

The Netherlands codes have implemented lower return periods when wind is the dominant load, for Class 1: 900 years, for Class 2: 1200 years and for Class 3: 1400 years. Higher risks and associated lower return periods are also used for natural hazards such as earthquakes. The widely used return period for earthquakes in earthquake sensitive areas is approx. 450 years.

We conclude that it may be necessary to accept for the Netherlands Antilles a lower figure than the return period of 1400 years for a design storm that is currently accepted in the Netherlands. Also, structural reliability classes (safely classes) may need to be defined. Any choice concerning the design return period and the classification of buildings should be based on consideration of the balance between the cost of investment in the buildings or retrofitting on the one side, and the risk in terms of damage potential (both human and economic) on the other side.

### *3.4 Calculation of design dynamic wind pressure*

Given the uncertainty in the adoption of a design wind load, as pointed out in the previous section, in this section we give a method of calculating the design dynamic pressure of the wind load irrespective of return period. From the Saffir/Simpson scale the calculation of the wind dynamic pressure over flat land at a height of 10 m is performed as follows.

*1) Conversion from 1 min average to hourly averaged wind speeds*

The wind speeds given in the Saffir/Simpson scale are transformed to hourly mean winds, by dividing them by a factor of 1.25.

*2) Calculation of mean dynamic pressure*

the dynamic pressure is calculated from the hourly mean wind speeds by:

$$p = \frac{1}{2}\rho v^2$$

where:

v is the hourly averaged wind speed, in m/s

ρ is the density of air, 1.25 $kg/m^3$

p is the mean dynamic pressure in N/m$^2$;

*3) Adding the influence of turbulence*

The dynamic pressure, associated with the occurrence of the maximum gust, is calculated by:

$$p_{dyn} = \frac{1}{2}\rho v^2 G$$

where:

G is a gust factor: For relatively flat terrain G = 2.20

$p_{dyn}$ is the dynamic wind pressure, including gusts, in N/m$^2$

Table 3.6: Dynamic wind pressure at 10 m height for various wind speeds.

| Saffir/Simpson scale | mean wind speed (m/s) | design dynamic pressure (N/m$^2$) |
|---|---|---|
| 1 | 42 | 1684 |
| 2 | 49 | 2293 |
| 3 | 58 | 3213 |
| 4 | 69 | 4545 |
| 5 | > 69 | > 4545 |

Although for terrain with increased roughness the dynamic pressure may be affected by the terrain roughness, for reasons of simplicity it is advised to use flat terrain conditions, as these normally lead to conservative wind loads.

The dynamic pressure given in Table 3.6 applies to a height of 10 m. For other building heights, the dynamic pressure is calculated from:

$$p_{dyn}(z) = \frac{v^2}{54}\left[\ln(\frac{z}{0.03})\right]^2\left(1+\frac{7}{\ln\frac{z}{0.03}}\right)$$

where

v is the hourly averaged design wind speed, in m/s

z is height above average terrain level, in m.

1+7/ln(z/0.03) The gust factor G as function of height z, for z = 10 m, G = 2.2, for z = 30 m, G = 2.0

When buildings are located on slopes of hills or on escarpments, an additional increase of wind speed may be needed to take into account. Reference is made to section 8.4 of ENV 1991-2-4.

### *3.5. Conclusions*

Currently, buildings in the Netherlands Antilles and Aruba are often designed using the values, given in de Netherlands wind loading code NEN 6702. At both the windward and leeward islands however, the hourly mean wind speed with a mean return period of 1500 years (which is the basis of the Netherlands code) is at the Netherlands Antilles and Aruba much larger than the value found in the Netherlands.

Application of the correct values for the hourly mean wind speed at the Netherlands Antilles and Aruba leads to design wind loads of a factor 4 higher for the Windward islands, compared to the calculation, using the Netherlands code. For the leeward side islands, the design load will be lower than on the Windward islands, but will also be higher than found with the Netherlands code, about a factor .

From these considerations it can be concluded that it is important to define a wind loading code for the Netherlands Antilles and Aruba, in which the adoption of a design wind speed is based on observations at the islands. Principal choices regarding return period and safety level need to be made to determine the values for the design wind loading on buildings and structures at the Netherlands Antilles and Aruba. These design loads will be different for the Windward and Leeward islands.

# 4 WIND LOADS ON BUILDINGS AND STRUCTURES

## *4.1 General*

From the considerations given in the previous chapters, we cannot establish a definite design load. In this report we have assumed the top of Saffir/Simpson scale 2 (49 m/s, 1 min. average) to be appropriate for the leeward islands, and the center of scale 5 (64 m/s, 1 min. average) to be appropriate for the windward islands ***as an example***. The estimated return period of these wind speeds is in the order of 100 years.

The dynamic pressure is calculated using the procedure in section 3.4. The values are (in rounded figures) 2100 N/m$^2$ for the Leeward islands and 3600 N/m$^2$ for the Windward islands.

Design wind loads are determined by multiplying the dynamic pressure by shape coefficients. These apply to the pressure outside of the building and inside the building. The total pressure on a wall or roof is:

$$p_{wall} = p_{dyn} \left(C_{p,ext} + C_{p,int}\right)$$

For normal, closed buildings the internal pressure coefficient can be taken between $C_{p,int}$ = + 0.3 and - 0.3. The external pressure coefficients $C_{p,ext}$ depend on the location at the building face. Near corners of the building, higher so-called local coefficients may apply.

## *4.2 Walls, windows, doors and wall-infills and cladding*

Shape factors are given in building standards for a wide range of building shapes and dimensions. The values for the shape factors for some areas on a building are:

Windward walls: $C_{p,ext}$ = 0.8 (pressure)

Leeward walls: $C_{p,ext}$ = -0.4 to -0.5 (suction)

Parallel walls: $C_{p,ext}$ = -0.9

Locally at upwind edges: $C_{p,ext}$ = -1.3

The local areas are the smallest of 0.1 times the height or 0.2 of the building width.

For more details on shape factors reference is made to (NBS, 1977), or to relevant wind loading codes such as (ANSI 58.1, NEN 6702, ENV 1991-2-4).

This means that on the windward side the pressure difference across the wall is in the order of 1.1 times the dynamic pressure. For parallel walls, this difference may amount to 1.2 times the dynamic pressure. In local situations, that is near upwind corners, a pressure difference of 1.6 times the dynamic pressure may act.

This leads to the following indications of maximum loads on walls:
- leeward islands: $1.2*2100 = 2520$ N/m$^2$
- windward islands: $1.2*3600 = 4320$ N/m$^2$

For wall cladding, windows and doors, a local wind load of $1.6 * 2100 = 3360$ N/m$^2$ or $1.6 * 3600 = 5760$ N/m$^2$ can be advised.

## *4.3 Roofs and roof cladding*

For flat and pitched roofs (roof angles up to 30$^o$) normally wind suction will act on the roof. As a reasonable guideline, it is advised:

*Flat roofs:*
- Area within one building height from the wall: $C_{p,ext} = -1.0$
  Remainder: $C_{p,ext} = -0.7$

*Pitched roofs:*
- wind parallel to the roof: design as flat roof
- wind perpendicular to the roof, windward face:
  . pitch > 10$^o$ $C_{p,ext} = -0.8$
  . pitch > 20$^o$ $C_{p,ext} = -0.4$
- wind perpendicular to the roof, leeward face:
  . $C_{p,ext} = -0.7$

These factors have to be multiplied with the dynamic pressure of 2100 and 3600 N/m$^2$ to find the design loading on the roofs.

# 5 RECOMMENDED MEASURES

## *5.1 Engineered buildings*

The most important recommendation is that buildings in hurricane prone areas preferably should be *engineered buildings*. This means that design calculations should be made on the basis of mandatory design wind loads and mandatory codes for taking into account the effect of wind loads and various other load combinations.

The essence of building against hurricanes is to withstand high uplift loads on roofs and the high loads perpendicular to walls. Also, due attention should be paid to infills of walls and windows and doors in walls. Very often, no specific requirements are imposed on these infills. For engineered buildings it is important that these infills are also capable of withstanding the design loads. Windows are particularly vulnerably to flying debris during hurricanes. It is therefore recommended that windows are equipped with shutters or temporary provisions that can protect them from flying debris during wind storms.

In this chapter, general recommendations for the design of buildings are given, based on knowledge from earlier events. These recommendations partly originate from (NBS, 1977).

## *5.2 Conceptual design.*

Gibbs (1996) has analysed damage caused by Hurricane Luis in Sint Maarten. He gives general recommendations for the design and construction of buildings. The most important factor determining success or failure of buildings in his opinion is the conceptual design. With respect to hurricanes, suitable design concepts are particularly important for light-weight structures (timber and corruated-metal walls and roofs).

Unfavourable features are:

- L-shaped plans
- Mono-pitched roofs
- Shallow-pitched gable roofs
- Long overhangs at the eaves and gables
- Long overhangs continuous with the main roof
- Corner balconies

Favourable features, which were found in Sint Maarten are:

- Compact plans
- Hipped roofs
- Steep-pitched gable roofs

- Short overhangs at the eaves
- Canopies discontinuous with the main roof
- Parapets

Generally, the details of the buildings determine the overall performance of buildings during hurricanes. He states that the connections are the most relevant part:
'the roof sheeting must be adequately connected to the purlins. The purlins must be adequately connected to the rafters, the rafters must be adequately connected to the wall plates. The wall plates must be adequately connected to the wall studs. The wall studs must be adequately connected to the base sleepers. The base sleepers must be adequately connected tot he base walls or piers. The piers must be adequately founded.'

Gibbs also gives general recommendations on the prevention of large damages caused by hurricanes. He recommends to improve the building standards, to prevent large damage, as observed on St. Maarten.

### *5.3 General recommendations for concrete and masonry buildings*

The following general recommendations for concrete and masonry buildings are given.

1. It is recommended to use a peripheral bond beam connecting all load-bearing masonry elements together. Alternate construction schemes include the use of a cast concrete beam and the use of bond beam masonry units through the top course. (Refer to figure 5.1 which gives a schematic view). Both schemes permit placement of horizontal reinforcement to achieve better integrity at the top. It should also be mentioned that a substantial bond beam relieves the walls from resisting forces in out-of-plane flexure and supplements the diaphragm rigidity of the roof.

2. Use of appropriate lintels and connections between lintels and the rest of the walls. Alternate cast-in-place and precast lintels are indicated in figure 5.1. A most effective means of achieving continuity of the lintels is to cast them integrally with the bond beam.

3. The roof shall have adequate diaphragm capability to transmit horizontal loads to shears walls below. Therefore, the roof system should preferably be braced in both directions

4. The use of slender vertical elements such as those built with single stacked concrete masonry units to provide columns for the support of large roof overhangs at re-entrant corners or landings should be avoided. This restriction is necessary because these elements are highly susceptible to impact by flying debris and uplift transmitted from the roof.

5. Large roof overhangs will lead to high uplift forces on the roof and therefore require more substantial construction provisions against high winds.

6. Overall integrity of the system should be assured by providing continuity between vertical wall reinforcement and the roof and foundation through appropriate utilisation of reinforcing bars and anchor bolts and by providing continuity between intersecting walls by means of tiebars and/or horizontal reinforcement grouted within a bond beam course and extending into the neighbouring walls and partitions. Figures 5.2 - 5.5 show typical details

7. The use of infill wall construction in building systems under a severe wind environment should be encouraged. The masonry infill walls should be laid first on top of a cast concrete wall footing and attached to it by means of dowels lapped at sufficient distance with the vertical reinforcement, placed into grouted cores of the masonry blocks.

Figure 5.1: Masonry bearing wall constructions (NBS, 1977)

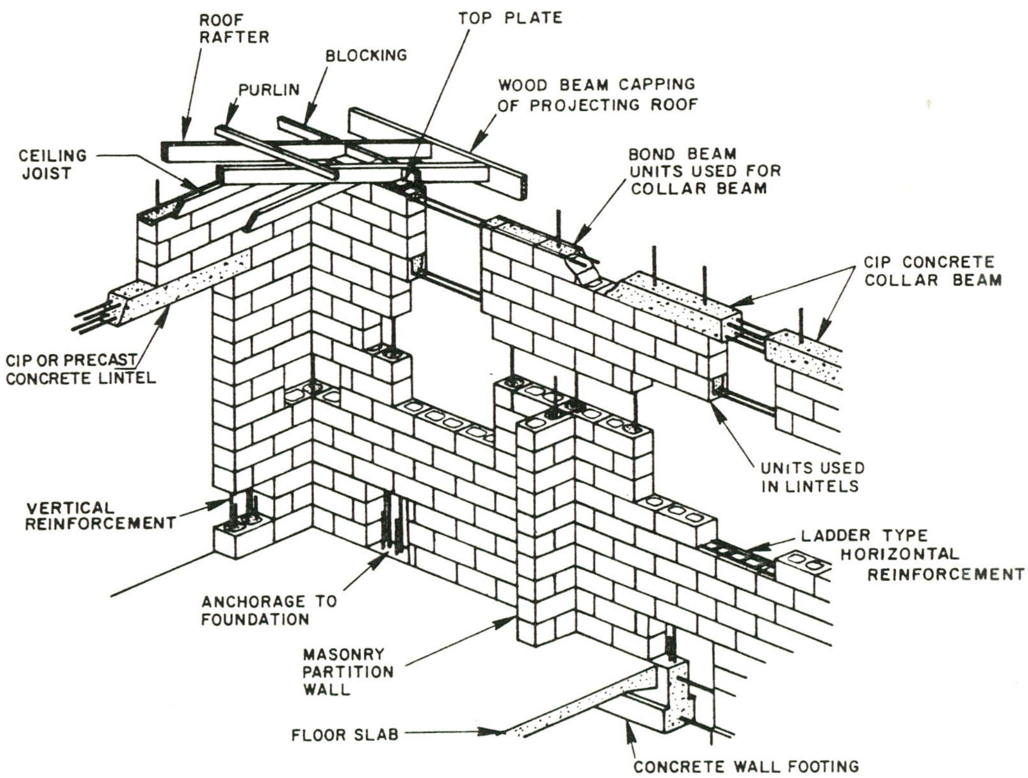


Figure 5.2: Joint reinforcement in masonry walls: (a) truss, (b) ladder, (c,d) prefabricated corner sections, (e,f) prefabricated tee section (NBS, 1977)

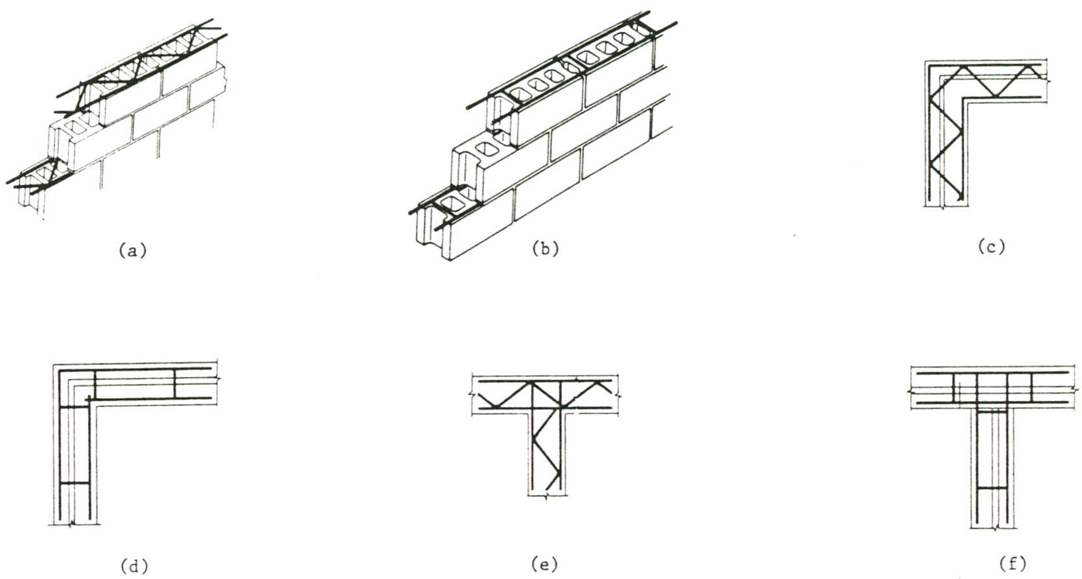



Figure 5.3: Horizontal reinforcement between intersection masonry walls: (a, b) without bond beams, (c,d) with bond beam (NBS, 1977)

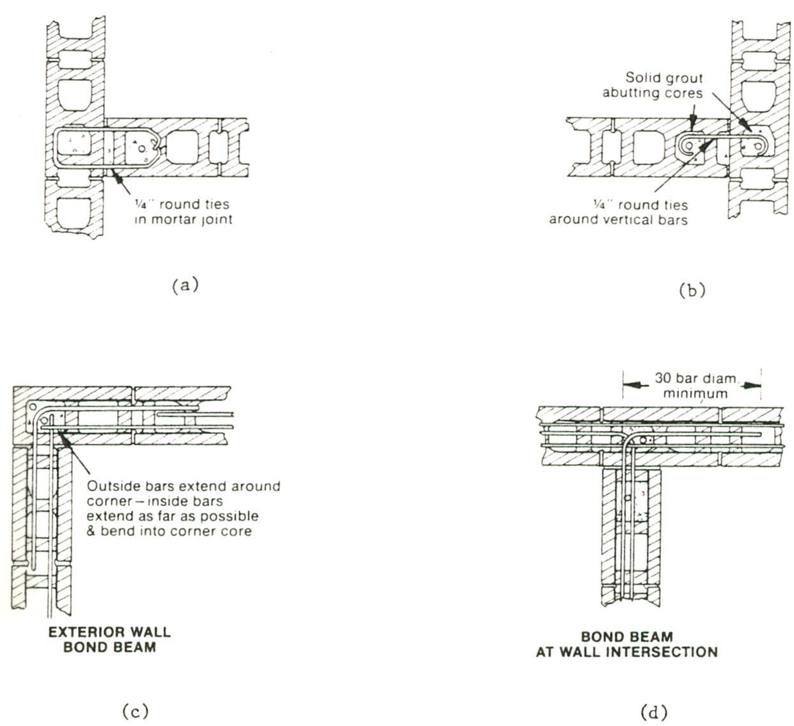


Figure 5.4: Application of metal anchors in masonry construction; (a) tiebar anchor to wood joist, (b) twisted anchor connecting wood joist to concrete masonry bearing wall, (c) tiebar anchor connecting concrete masonry intersecting walls. (NBS, 1977)

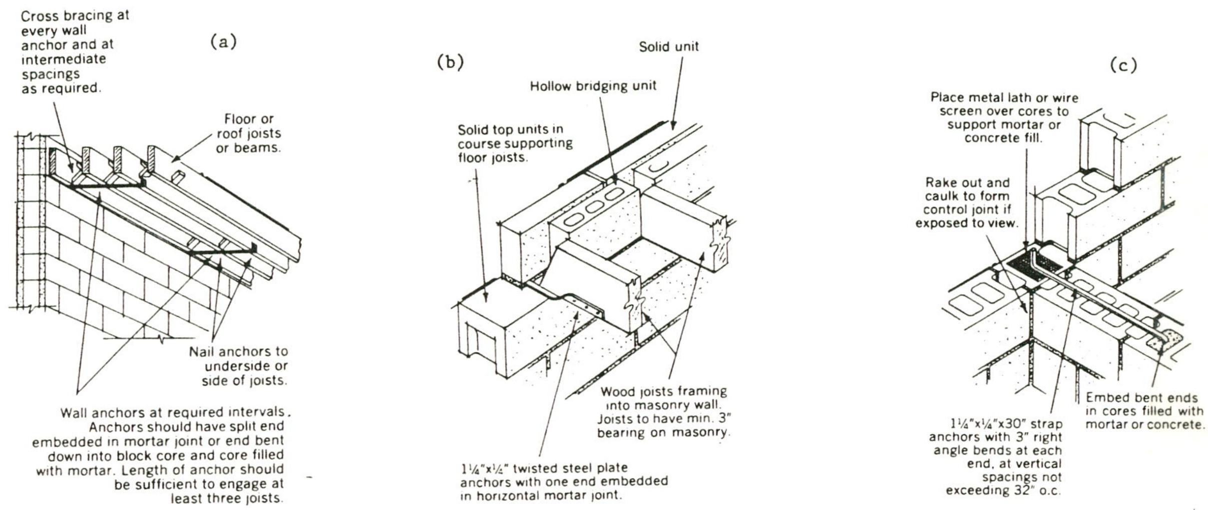
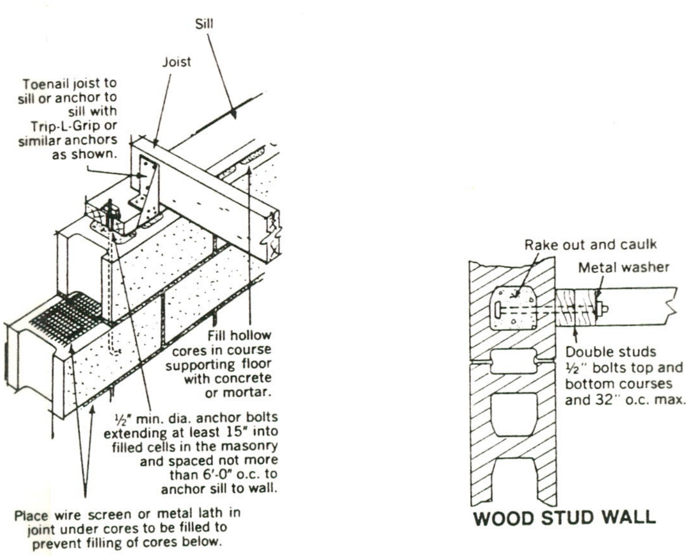



Figure 5.5: Application of anchor bolt for masonry connection: (a) connection of sill plate to concrete masonry bearing wall, (b) connection of concrete masonry wall to wood stud wall (NBS, 1977)




(a) (b)

### *5.3 General recommendations for wooden buildings*

The wood frame must not only be anchored to the foundation at its base, but must also provide a continuous tie from the foundation to the roof. Anchorage of wall framing to a concrete foundation may be accomplished with hooked bolts embedded 0.2 to 0.3 m in the concrete. A large washer should be used to spread the load on the wood framing, refer to figure 5.6

The roof must be tied together as well as anchored. Panel type covering material nailed to the frame provides this continuous tie. Roof framing is effectively tied to walls with sheet metal brackets or wood cleats. Figure 5.7 gives a typical timber wall construction. Where a ceiling joist-and-rafter system is used, sheet metal straps nailed to joists and studs provide a good tiedown for roof framing. It is advised to used galvanised steel straps.

The use of engineered roof trusses is strongly recommended. Special attention is to be paid to the fixing of trusses to the walls to prevent uplift forces.

Figure 5.6: Anchor bolt through concrete block foundation and connecting between wall and bottom plate (NBS, 1977)

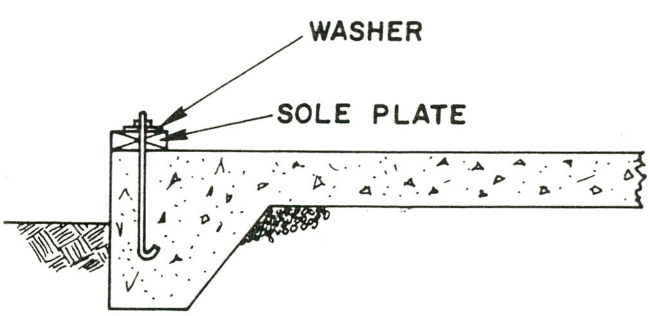
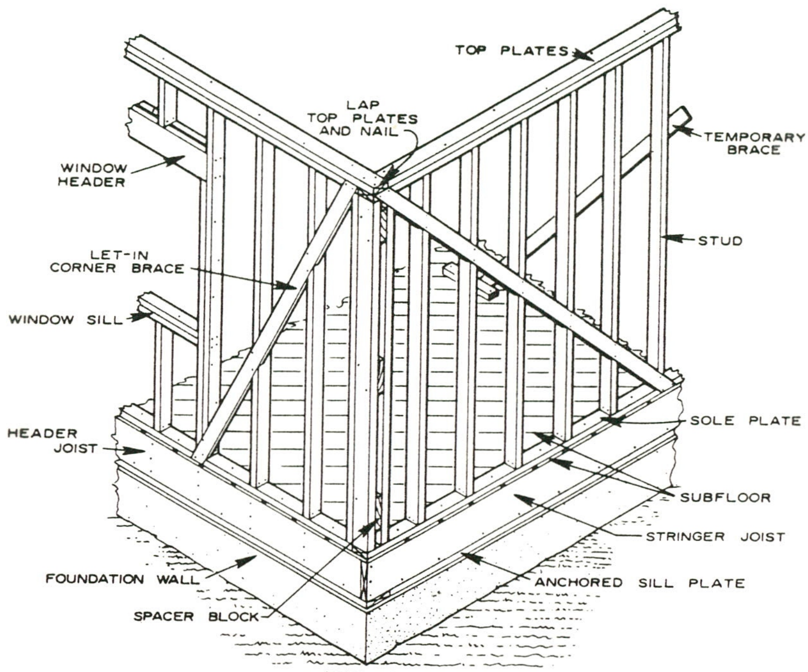


Figure 5.7: Typical timber wall construction (NBS, 1977)



### *5.4 Recommendations for roof covering materials*

Flying debris is one of the main causes of hazardous situations during hurricanes and is also responsible for a large amount of secondary damage. Roof coverings such as roof tiles should be fixing with special fixing elements. Most types of roof tiles have special fixings that are commercially available. For test methods of the wind resistance of roof coverings, refer to NEN 6707 and NPR 6708. Also, the fixings of roof coverings consisting of corrugated steel sheets must be designed to withstand extreme wind loads.

### *5.5 Recommendations for windows and doors*

Most commercially available window and door-systems have a specified design pressure and have been tested to withstand that pressure. It is recommended to check windows and doors for this design pressure. Reference is made to the testing procedures given in NEN 3660 and NEN 3661. It is remarked that the required load levels in these codes are not appropriate for the Netherlands Antilles and Aruba in hurricane conditions, such as given in chapter 4, but the test method may be applied.

Damage to windows can have a serious impact on the wind pressure acting in the building, specifically for windows on the windward side of a building. To avoid damage to windows due to wind or due to flying debris, it is recommended to protect them during wind storms with (temporary) shutters. Due attention should be paid to the strength of the fixings of these shutters. Gibbs (1996) also recommends the use of laminated glass, which after failure will exclude the weather during the hurricane.

# 6 CONCLUSIONS AND RECOMMENDATIONS

In this report design wind loads which may be adopted in the Netherlands Antilles and Aruba for the case of hurricanes have been discussed. The study leads to the conclusion that hurricane wind loads are the dominating wind loads for both the Windward islands and the Leeward islands. This is despite of the rare occurrence of hurricanes in the vicinity the Leeward islands.

It was found that using the Dutch wind loading code leads to design wind loads that are far too low for the situation on the Netherlands Antilles and Aruba. It is therefore strongly advised to develop a building standard, based on the local wind field data.

The historic hurricane wind speed statistics need to be investigated further, in order to establish the probability of exceedence of extreme wind speeds more accurately. The current analyses can also be refined based on measured wind speeds (meteorological data) or based on Monte Carlo simulations.

Since hurricane wind loads will have a large impact on the design of buildings and structures, which may have serious implications on the cost of buildings, it is recommended to conduct a risk analysis and to decide on target reliability levels for structures. The use of safety classes for buildings should be considered. This offers the possibility of adapting the design hurricane wind load to the importance and use of the building.

On the basis of the target reliability of buildings and structures, target return periods for extreme wind speeds need to be established for design calculations of buildings and structures. It is advised that the authorities responsible for building regulations and building safety prescribe mandatory hurricane wind loads for the design of buildings and structures for both the leeward and windward islands. In addition, it is recommended to prescribe the use of a specific wind loading code as mandatory.

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# Jähnichen, Tamara

**From:** Geurts, Chris
**Sent:** Wednesday, September 25, 2002 4:32 PM
**To:** Jähnichen, Tamara
**Subject:** RE: Rapport 96-CON-R0781-3

Laat die student mij even bellen. Ik wil graag weten wat hij ermee kan doen, en wellicht kan ik hem beter persoonlijk helpen dan met een rapport.

groet

Chris

-----Original Message-----
**From:** Jähnichen, Tamara
**Sent:** 25 September 2002 14:28
**To:** Geurts, Chris
**Subject:** Rapport 96-CON-R0781-3

Hallo Chris,

Een student van de TU Eindhoven heeft rapport 96-CON-R0781-3, Hurricane Hazard at the Netherlands Antilles and wind loads on buildings, aangevraagd. Het is een rapport van 4 december 1997. De opdrachtgever was het Nederlandse Rode Kruis afd. Bonaire, Kaya Playa Lechi 8, Kralendijk, Bonaire, N.A.. Is dit rapport voor derden beschikbaar?

Alvast bedankt voor je antwoord en groetjes van Tamara

**Jähnichen, Tamara**

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**From:** Yuri Daal [YawningYak@hotmail.com]
**Sent:** Monday, September 23, 2002 2:12 PM
**To:** wegwijzer@tno.nl
**Subject:** TNO bouwrapport windsnelheid

Goede dag,
ik ben een student op de T.U. eindoven. Ik wilde weten hoe ik aan een oude TNO-bouw rapport kan komen. Namelijk;
96-con-R0781-3, dec. 1997. (windsnelheid)

vriendelijk groet,

Yuri Daal