Physical Flood Vulnerability of Residential Properties in Coastal, Eastern England

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Declaration

This dissertation is the result of my own work and includes nothing which is the outcome of work done in collaboration. The work for this dissertation has not been and is not being, in part or wholly, submitted for another degree, diploma, or similar qualification. This dissertation does not exceed 80,000 words in length, excluding the bibliography.

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Summary

This study demonstrates that significant improvements could and should be made in the manner in which society manages its risk to natural disasters. The focus is on managing the physical vulnerability of residences (houses and flats) to flood disasters in coastal, eastern England. To complement the extensive knowledge of damage from slow-rise flood depth, this study examines the lateral pressure from flood depth differential between the inside and outside of a residence and flood velocity.

To determine the impacts on residences in coastal, eastern England, Canvey Island on the Thames Estuary and Kingston-upon-Hull on the Humber Estuary were selected as case study sites. Field surveys determined characteristics of the physical vulnerability of residences in these locations to floods. A first-order analysis indicated the failure modes of most prominent concern to be analysed in detail:

- The rate of rise of flood water inside a residence, to establish that pressure differentials could damage the residence.
- Analysis of glass failure, focused on large, low units in doors.
- Analysis of wall failure, focused on cavity walls of unreinforced masonry.

The observations and calculations were applied to developing a new form of vulnerability profiling: two-dimensional "vulnerability matrices" with flood depth differential along one axis, flood velocity along the other axis, and the matrix cells displaying a damage outcome.

The vulnerability matrices were used to illustrate possible disaster management tools. The "loss equivalent percentage" method of describing quantitative risk was presented for simulated storm surge scenarios on Canvey Island. Qualitative decision-making tools and analysis strategies for residence flood vulnerability management were also discussed.

The results apply specifically to modern residences in England because those were the residences observed in the field surveys. The techniques and methods nevertheless could be used for similar analyses for other locations, other residence types, and other disasters. This study provides new knowledge and methods which contribute to understanding, describing, and managing society's risk to natural disasters.

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Acknowledgements

Alexander Pope wrote "A little learning is a dangerous thing; drink deep, or taste not the Pierian spring: there shallow draughts intoxicate the brain, and drinking largely sobers us again". While it would seem that a Ph.D. is the ultimate in which to "drink deep", it involves only three years of work and a handful of minds. I have barely tasted the proffered riches. So much has been found, but so much is yet to know.

Alfred Tennyson wrote "To strive, to seek, to find and not to yield". The uncountable dimensions and vast networks of "so much yet to know" are never depressing, instead presenting an evolving challenge, confronted by a thirst which can only ever be partially satiated. The desire for knowledge may never be entirely fulfilled, but we must nevertheless strive, seek, and find ways to continue the likely-vain efforts to not only drink deeply but to realise that we have indeed drunk deeply without becoming mired in self-admiration. Tennyson's phrase might better have read "To strive, to seek, to find, but never to yield" to either despair or to the temptation to have complete faith in our own abilities.

I thus dedicate this work to the humility of knowledge, the perseverance it creates, and the awe it inspires.

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Any errors or misapprehensions are my fault.

Symbols and Abbreviations

This dissertation uses only this list of global variables. No local variables are used or defined, hence variables are not listed after each equation. Instead, this list should be used. Units and SI prefixes are not defined as they are standard practice. Mathematical symbols are not defined because they also follow standard practice, including the use of the differentials d and ∂. Some subscripts not listed here are used, but they are full words, so the context makes their meaning evident.

Latin Alphabet

A B C D E F G H I J K L nodes on a wall panel (Chapter 9)

A B C D E F G H I J K L M N O P Q R glass panes of different dimensions (Chapter 10)

A .....residence plan area [m^2]

avg .....(as a subscript) average value expressed as the arithmetic mean

b .....height above ground of the bottom of a residence component [m]

C .....coefficient for the power law in crack flow [$m^3\ s^{-1}\ Pa^{-n}$ per m, per m^2, or as is depending on residence component being considered]

C_{drag}.....drag coefficient [$m^{1-2n}\ s^{2n-1}$ per m, per m^2, or as is depending on residence component being considered]

C_f.....compression depth factor, i.e. the percentage by which the effective thickness of the panel is reduced due to crushing at the edge of the masonry when rotation occurs [no units]

DS .....level of the damage scale $\in$ {0, 1, 2, 3, 4, 5} and reported as DS0, DS1, DS2, DS3, DS4, or DS5.

E E_x E_y .....Young's modulus (modulus of elasticity), respectively in general, in the x-direction and in the y-direction [Pa]

EA.....Environment Agency

exp().....the exponential function, i.e. the argument in parentheses becomes the exponent of the natural base of logarithms e = 2.7182818284…

EXP[f(x)].....The mathematical expectation of f(x) [same units as f(x)]

f.....flood depth [m]

$f(x_r)$ .....A real function on a random variable [any units]

f_{diff}.....flood depth differential between the inside and outside of a residence [m]

FHRC.....Flood Hazard Research Centre

FIR.....Flood Infiltration Rate [$m^3\ s^{-1}$]

FRR.....Flood Rise Rate [$m\ s^{-1}$]

g .....acceleration due to gravity [9.80665 m s^{-2}] (Lide, 2001)

$h(f_{diff},v)$ .....percent of residential area which experiences each f_{diff}-v combination for a given flood scenario [no units]

H .....height of residence component (e.g. wall or door) [m]

HGISL .....Halifax General Insurance Services Ltd.

j .....number of storeys or levels in a residence [no units]

k .....parameter for Wei(ξ)

L.....characteristic length of crack used for calculating Re [m]

LEP .....loss equivalent percentage [no units]

$LEP(A,j,f_{diff},v)$ LEP for a specific residence defined by A and j at the given f_{diff}-v combination [no units]

$LEP(f_{diff},v)$ ....LEP across a residential area assuming that the entire residential area is inundated by the given f_{diff}-v combination [no units]

m .....ultimate resisting moment per unit length [N]

max .....(as a subscript) maximum

min .....(as a subscript) minimum

min() .....minimum of the arguments in parentheses

M M_x M_y.....bending moment, respectively in general, in the x-direction, and in the y-direction [Nm]

n .....exponent for the power law in crack flow [no units]

ODN .....Ordnance Datum Newlyn, the U.K.'s national coordinate system for heights above mean sea level.

$P[f(x_r)]$ .....the probability distribution of $f(x_r)$ defined appropriately if $f(x_r)$ is continuous or discrete) [no units]

ΔP .....pressure difference [Pa]

$\Delta P_{p\%}$.....pressure difference for a glass failure probability of p% [Pa]

$\Delta P_{y=y1}$ .....pressure difference at the point y = y1 [Pa]

Q .....volumetric flow rate [m^3 s^{-1}]

Q_{crack}.....volumetric flow rate per length for cracks [m^3 s^{-1} per m] or per area for walls [m^3 s^{-1} per m^2]

r.....correlation coefficient [no units]

R .....area of residence component or crack [m^2]

RC(A,j) .....percent of each A-j residence class [no units]

Re.....Reynolds number [no units]

s.....height above ground of the top of a residence component [m]

S .....parameter for Wei(ξ) [N m^{-4}]

t .....time [s]

USACE .....United States Army Corps of Engineers

v .....velocity [m s^{-1}]

w1 .....residence configuration with 1 exterior wall [no units]

w2adj .....residence configuration with 2 adjacent exterior walls [no units]

w2opp .....residence configuration with 2 opposite exterior walls (e.g. terraced middle) [no units]

w3 .....residence configuration with 3 exterior walls (e.g. semi-detached) [no units]

w4 .....residence configuration with 4 exterior walls (e.g. detached) [no units]

W .....width (or length) of residence component (e.g. wall, wall panel, door, or crack) [m]

Wei(ξ) .....Weibull distribution over the parameter ξ [no units]

x .....coordinate direction

x_r .....random variable [any units]

y .....coordinate direction

z .....coordinate direction

Greek Alphabet

α.....coefficient for quadratic formula in crack flow [kg m^{-4} s^{-1}]

β.....coefficient for quadratic formula in crack flow [kg m^{-7}]

γ .....mass supported by wall panel [kg]

δ .....deflection (of masonry slabs or a glass pane before failure) [m]

Δ.....difference

ζ .....factor in yield line analysis [no units]

η .....factor in yield line analysis; for a residence wall η = H/W [no units]

θ θ_{x1} θ_{x2} θ_y.....angles of rotations in slabs for yield line analysis [radians]

λ .....fraction of residence's dead load supported by a wall panel [no units]

μ.....dynamic viscosity [Pa s]

ν .....Poisson's ratio [no units]

ξ .....parameter for Wei(ξ) [no units]

π .....the ratio of a circle's diameter to its radius, π = 3.1415926535…

Π .....external residence perimeter [m]

ρ_b.....density of material used in cavity walls [kg m^{-3}]

$\rho_{brickwork}$ .....density of brickwork [kg m^{-3}]

$\rho_{blockwork}$.....density of blockwork [kg m^{-3}]

ρ_w.....density of water [kg m^{-3}]

σ.....bending stress or tensile stress induced by an applied pressure [Pa]

$\sum$.....sum

φ.....factor in yield line analysis [no units]

ψ.....friction coefficient to determine the ultimate resisting moment perpendicular to the bed joint in yield line analysis [no units]

Ω.....the material property of flexural or tensile strength in glass [Pa]

Other

þ.....thickness of a material or residence component [m]

$þ_{brickwork}$.....thickness of brickwork [m]

$þ_{blockwork}$.....thickness of blockwork [m]

$þ_{glass}$.....thickness of a glass pane or a sheet of glass [m]

Ч.....factor for forces on a structure due to deposited sediment [no units]

Part I

1. Coastal Settlements at Risk

1.1 Background and Justification

Society's vulnerability to natural hazards exacts a heavy toll. EM-DAT (2002) catalogues approximately 380,000 deaths and more than 1 billion people affected—injured, made homeless, or requiring assistance—by natural disasters from 1997 to 2001. IFRC (2002) notes that despite the apparent trend of decreasing deaths from natural disasters, lives and property are increasingly exposed to the potential for natural disaster losses.

Rising economic losses from natural disasters are also apparent. Munich Re (2002) reports that the direct economic cost of natural disasters, in 2001 values, increased from US$42.2 billion from 1950 to 1959 to US$652.3 billion from 1990 to 1999. This increase is marked despite concerns about biases in the figures due to:

- an increase in world population during this time period;
- more comprehensive reporting of economic losses in later years; and
- a single event skewing the 1990-1999 figure—the earthquake near Kobe, Japan on 17 January 1995 caused approximately one-sixth of the total losses for the 1990s.

Insured losses from natural catastrophes are large too: US$10 billion in 2001, slightly below the 15-year average of US$14.3 billion per year adjusted to 2001 figures (Swiss Re, 2002).

The significant impacts of society's vulnerability to natural hazards are evident in the U.K., especially regarding economic losses. Examples of major events between 1987 and 2000 are provided for England in Table 1.1.1. Table 1.1.1 lists only hydrometeorological events, i.e. disasters involving hydrological and meteorological processes such as floods and wind storms, because they have tended to cause the most fatalities in the U.K. in recent centuries. Nonetheless, England and the U.K. experience many other natural hazards including:

- earthquakes (Musson, 1996);
- landslides (Lee, 1995);
- subsidence and erosion in the short-term and long-term (ABI, c. 1999; Dunham and Gray, 1972);

- pathogenic microbes such as Foot-and-mouth Disease across Great Britain in February 2001 and Legionnaires' Disease in Barrow-in-Furness, Cumbria, England in August 2002; and
- larger, hazardous biota such as jellyfish.

Some of these natural hazards are linked to hydrometeorological phenomena, such as rainfall-induced landslides, storm-exacerbated coastal erosion, and localised subsidence due to temperature and precipitation changes.

Table 1.1.1: Examples of Natural Disasters in England

Date	Event	Casualties	Direct Damage (value at the time of the event)	Principal Sources
15-16 October 1987	Windstorm in southern England.	19-20 dead	US$1.7 billion.	EM-DAT (2002) Ogley and Reynolds (1989)
25 January 1990	Windstorm in southern England.	45 dead	UK£150 million.	EM-DAT (2002)
9-15 April 1998	Floods in middle England.	5 dead	UK£400 million.	ODPM (2001)
25 December 1999	Storms and storm surge in southern England.	8 dead	No information.	EM-DAT (2002)
October to December 2000	Freshwater floods and storms throughout the U.K.	At least 2 drownings and at least 16 other deaths.	Insurance cost of UK£1.3 billion.	ABI (2001) EA (2000) EM-DAT (2002)

The high impact of flood events in England, such as the disasters in Table 1.1.1, led MAFF (2000) to estimate the property and land at risk from flooding in England. Based on this report, ODPM (2001) summarises the flood risk:

> Approximately 10,000 km^2 or 8% of the total area of land in England is at risk from river flooding, including tidal rivers and estuaries. Approximately 30% of the coastline is developed and some 2,500 km^2 of land (1.5% of the total area) is at risk of direct flooding by the sea. As a result, about 1.7 million homes and 130,000 commercial properties worth over £200 billion and…agricultural land worth about £7 billion are at risk from flooding. This equates to about 10% of the population and 12% of the agricultural land.

Coastal, eastern England is particularly vulnerable to a range of flood types. Freshwater flooding triggered by snowmelt caused devastation in March 1947 (Barker, 1948) whereas

freshwater flooding from rainfall affected vast parts of the region in Autumn 2000 (Table 1.1.1), in February 2001, and in October 2001. Storm surge flooding is a major concern in this region. A storm surge is an increase in sea level caused by winds blowing from the sea to the shore combined with low atmospheric pressure over the sea. If the surge coincides with high astronomical tides, water levels can be several metres above the normally-expected maximum tide level. This problem in coastal, eastern England is exacerbated by long-term subsidence which is increasing the relative level of the sea to the coast by almost 4 mm per year (HMSO, 1998).

An example of a major storm surge disaster in coastal, eastern England is the North Sea storm surge of 31 January to 1 February 1953. In the U.K., mostly in eastern England, this event killed more than 300 people and caused approximately UK£5 billion (1998 value) of damage (HMSO, 1998). More than 32,000 people were evacuated, 24,000 houses were damaged, 46,000 livestock were killed, 160,000 acres of agricultural land were flooded with salt water, and major transportation links were impassable (Summers, 1978). Other storm surges caused damage in eastern England on 11-12 January 1978 (Harland and Harland, 1980) and 18-19 September 2001.

Following the 1953 storm surge, the sea defences along England's east coast were raised and strengthened (Grieve, 1959). Further changes to defences occurred at more localised levels. For example, the construction in London of the Thames Barrier which opened in 1984 involved strengthening many sea defences along the Thames Estuary downstream from London (Thames Barrier and Thames Water, 1986).

MAFF (1994) and NAO (2001) catalogue deficiencies in England's coastal defences and inland flood defences respectively. The flood events in Table 1.1.1 have raised the public and political profile of flood issues so that significant, rapid changes are ongoing to the government authorities in England responsible for flood management. Some examples are:

- Shifting from individual, local schemes for flood defence to a catchment-wide approach: "Catchment-wide flood management plans are now being prepared for all 80 catchments in England and Wales" (Evans *et al.*, 2002).
- Moving the emphasis from developing design standards in structural flood defence to integrated flood risk management systems (Sayers *et al.*, 2002).

- Implementing a revised system in March 2002 for prioritising capital flood defence works which is more comprehensive in quantifying costs and benefits of proposed schemes (DEFRA, 2002).
- UK£100 million of investment between 2002 and 2012 for flood warning systems, including education and awareness campaigns, in England and Wales (Purnell, 2002).

These changes factor in the growing realisation that structural defences play only a small role in an effective long-term flood management strategy (Etkin, 1999; Kelman, 2001; Mileti *et al.*, 1999) and awareness of the effects of potential climate change (e.g. Hulme *et al.*, 2002).

Climate change refers to "statistically significant variations that persist for an extended period, typically decades or longer" in "classical measures of climate (e.g. temperature, precipitation, sea level, plus extreme events including floods, droughts, and storms)" (IPCC, 2001). Climate change is expected to affect floods in eastern England by (Hulme *et al.*, 2002):

- Increasing total winter precipitation and the frequency of extreme winter precipitation events but producing drier summers across the U.K., with eastern England experiencing some of the biggest relative changes.
- Increasing sea level in southeast England by between 0.26 m and 0.86 m above the 2002 level by the 2080s.
- Increasing the frequency of severe storm surge events, particularly on the east coast of the U.K., and permitting waves with greater energy to reach the shoreline.

Flooding is currently the biggest climate change issue for U.K. insurers (Crichton, 2001a). The U.K. insurance industry has become increasingly involved in flood management issues due to their concern about rising flood losses. Crichton (2001b) writes:

> Since 1961, the insurance industry has voluntarily guaranteed availability of flood insurance for all domestic residences in Britain [but in January 2001] announced that unless tighter planning guidelines were introduced, together with measures to improve flood defences, the flood insurance guarantee would expire at the end of 2002 for existing, as well as new properties.

As of August 2002, this issue remains unresolved (see also the original policy statement by ABI (2001)). The U.K. insurance industry's suggestions for properly managing flood risk include simplifying the institutional and decision-making arrangements for U.K. flood management, redefining the quantitative measure of acceptable flood risk for different types

of properties, and spending at least UK£145 million a year more on flood defences in England and Wales (ABI, 2001; ABI, 2002).

Concerns of many stakeholders merge in eastern England due to the potential devastation from a storm surge similar to the 1953 and 1978 events. MAFF (2000) indicates that the region between the Humber Estuary and the Thames Estuary, excluding London, does not contain as much property at risk from flooding as some other regions in England. Nevertheless, this area has UK£10.7 billion of property at risk from fluvial flooding and UK£9.7 billion of property at risk from sea flooding (MAFF, 2000). Internationally-recognised nature areas, such as the Wash and Breydon Water, are present along this region's coast (Ramsar, 2002) along with important historical sites, such as Landguard Fort near Felixstowe, Suffolk.

Continued development in eastern England has the potential to increase flood risk and much of this development occurs in coastal areas. Baxter *et al.* (2001) note that between 1951 and 1991, the U.K.'s population increased 12% but the population of many coastal areas in eastern England increased between 17% and 92%. Between 2001 and 2021, the coastal counties from Essex on the Thames Estuary to Humberside on the Humber Estuary are expected to gain 322,000 new households, 10% of England's projected need (ODPM, 1999). Kelman (2001) and Penning-Rowsell (2001) articulate concern that new developments will factor in neither the current flood risk nor how the new developments themselves may augment that flood risk through, for example:

- reducing the area of wetlands and flood plains which act as water storage areas;
- accelerating surface water run-off and directing it immediately into rivers, thereby rapidly increasing flow rates and river levels following rainfall; and
- changing flow patterns so that the speed of inundation, flood velocities, and flood durations increase.

Evidence that such phenomena manifest is from the October 2000 floods in Lewes and Uckfield in southeastern England (BB&V, 2001). Other changes in land use practices, such as cultivation techniques, have the strong potential to increase flood risk and the types of damage experienced during a flood event (Boardman, 2001).

Given this wide range of issues, concerns, and changes, improved quantification of flood risk in the U.K., including storm surge on the east coast of England, is important for the insurance industry, government, and individual and corporate property owners. Technically-detailed

research in this academic field and geographic area is needed. Work should cover all aspects of flood management in coastal, eastern England, including modelling a flood event starting at its meteorological origins, critiquing the existing institutional structures, and estimating the direct losses to insurers and the indirect losses to society due to a flood disaster.

1.2 Coastal Settlements at Risk Project

To contribute towards some of the issues identified in Section 1.1, a research project entitled "Coastal Settlements at Risk" was funded by Halifax General Insurance Services Ltd. (HGISL) (Kelman *et al.*, 2002; Thomalla *et al.*, 2002). This project was created to examine flood risk in coastal, eastern England. HGISL was primarily interested in residential properties in the coastal area from the Thames Estuary to the Humber Estuary (for a map, see Figure 4.1.1), thereby focusing this study immediately. This dissertation and Brown (2003) comprised the core research of the "Coastal Settlements at Risk" project.

The aim was to make a significant contribution to the understanding of flood risk to the built environment, especially residential properties, in coastal, eastern England. Brown (2003) researched, set up, and ran hydrodynamic models to simulate flooding from a storm surge in the case study site (Chapter 4) of Canvey Island. The results and outputs from his work are referenced and used in this dissertation where appropriate, mainly in Chapter 13.

The work described in this dissertation (see Section 1.3) tackles quantitative flood vulnerability assessment and formulates disaster management strategies. The quantitative and qualitative description of the flood risks faced by communities in coastal, eastern England, along with recommendations on the management of those risks, play a prominent role in the outputs from this work. Throughout such investigations, viewing flood risk in the wider context of risks from other natural hazards is helpful to achieve a complete understanding of the risks affecting residences. This study mentions non-flood hazards when appropriate, but otherwise, the focus is on floods.

1.3 This Dissertation

1.3.1 Objectives

The overall objective of this dissertation, representing an original contribution to research, is to demonstrate that significant improvements could and should be made in the manner in which society manages natural disaster risk. More specifically, the subobjectives are:

- To demonstrate this objective for the particular case of flood damage to residences in coastal, eastern England.
- To better describe, quantitatively and qualitatively, the risks for this case.

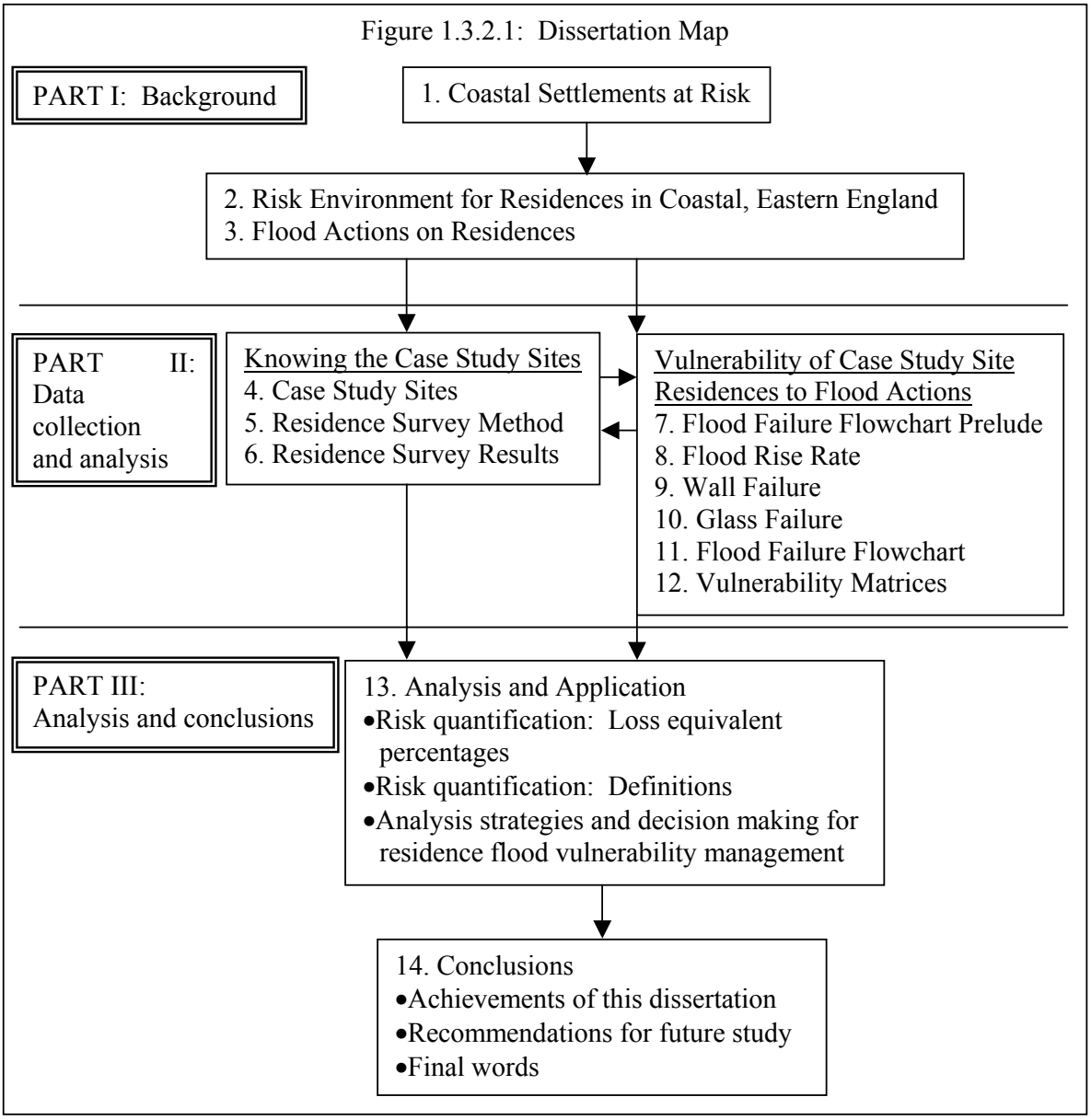
In order to meet these objectives, this dissertation illustrates for coastal, eastern England:

- The significance of flood depth differential (f_{diff}) and flood velocity (v) in causing flood damage to residences.
- The importance of structural failure in residences for overall flood damage.
- The improvement of the profiling of physical vulnerability for residences at risk from flooding.
- The usefulness of first-order calculations with respect to the first two points, i.e. residence structural failure during floods, to achieve the third point, i.e. improved vulnerability profiling.

1.3.2 Structure

Figure 1.3.2.1 illustrates the structure and flow of this dissertation. Progress through this dissertation is not linear, even though the Chapters must be presented in this fashion. Part II is divided into two sections which were completed in parallel although the Chapters in the left-hand block (Chapters 4-6) appear in this dissertation prior to the Chapters in the right-hand block (Chapters 7-12). Frequent exchange and cross-referencing occurs between the two blocks in Part II.

Figure 1.3.2.1: Dissertation Map



The themes of each part are:

Part I: Set the stage: •Reasons for the research.

•Starting point.

Part II: Core research: •Vulnerability profiling.

Part III: Use and Outputs: •Risk profiling and understanding.

•Policies and actions.

1.3.3 Preview

With reference to Figure 1.3.2.1, Part I of this dissertation sets the stage for the "Coastal Settlements at Risk" project and for this dissertation. The reasons for tackling the research and the starting point are provided. The geographic focus is identified as coastal, eastern England from the Thames Estuary to the Humber Estuary. The topical focus is identified as the physical vulnerability of residences to flood risk. The issue of flood risk is further scoped to examining damage resulting from the hydrostatic pressure imposed by the flood depth differential (f_{diff}) between the inside and the outside of a residence and the hydrodynamic pressure imposed by flood velocity (v). Damage due to water contact from slow-rise flood depth (f) must be considered too. This risk and this region may not be the predominant concerns for all sectors in disaster management in England or internationally, but a high level of vulnerability exists and much may be learned through selecting this focus and by examining the issues in detail.

Part II presents the core, new research of this dissertation. A new method for profiling the physical vulnerability of residences is developed and applied. Specific case study sites are selected as Canvey Island on the Thames Estuary and Kingston-Upon-Hull on the Humber Estuary. Field surveys yield the vulnerability characteristics of residences in these locations. In parallel, the principal failure modes of residences subjected to the identified flood risks are illustrated as flood water infiltration into a residence, glass failure, and wall failure. These three modes are quantitatively analysed resulting in vulnerability matrices. The vulnerability matrices suggest a failure mode for a given residence type which experiences a given f_{diff}, to the nearest 0.5 m, and a given v, to the nearest 0.5 m/s.

Part III describes the use of the vulnerability matrices in practice for specific scenarios. Outputs include loss scenarios for a storm surge on Canvey Island, risk profiles, and decision-making tools. Policies and actions are suggested for improving the manner in which society manages its risk to natural disasters, particularly with respect to flood management in coastal, eastern England.

1.4 Terminology and Context

Specific terminology must be defined and placed in the context of this study. The approach selected is to aim for functional rather than legalistic definitions. The definitions provided are

indicative and conceptual rather than rigid or specific enough to be devoid of intuitive understanding.

The terms “hazard”, “vulnerability”, and “risk” are clarified and specified for this study in Chapter 2. “Storm surge” was defined in Section 1.1. “Flood” generally refers to a flood disaster, an event during which uncontained liquid water detrimentally affects society such as through loss of life, disruption of normal activities, or damage to property.

A “residence” refers to a residential property or dwelling; i.e., a structure used for living or domestic purposes. Residences comprise houses and flats (apartments). “Residence components” are constituent parts of a residence such as ceilings, doors, floors, roofs, walls, and windows. Residence components may have “subcomponents” such as frames, glass, hinges, and locks on doors or specific timber boards forming a floor. The distinction between a residence component and a subcomponent is not necessarily definitive.

The term “residence” may encompass inanimate “contents” such as furniture, cutlery, toys, plants, and appliances, but not animate constituents such as people or pets. Systems such as electricity wires, water and wastewater pipes, and telephone cables are included. Vehicles and affiliated residence accessories, such as external swimming pools, sheds, and the grounds around a residence, may be included.

“Damage” is detrimental change. Damage includes the subset of “failure” which is catastrophic change; i.e., when restoring the original state is infeasible or undesirable, perhaps due to the amount of resources which would be required. Non-failure damage (non-catastrophic change) implies that expending the resources required for restoration to the original state is deemed acceptable. Although all damage is implied to be detrimental, any damage scenario yields opportunities, such as for the providers of goods and services to repair the damage.

Therefore, the judgement that damage is detrimental is subjective and this interpretation may vary amongst stakeholders or with the spatial and temporal scales considered. In considering residences, “damage” effectively describes the reduction in value sustained by an item or property whereas “loss” effectively describes the cost incurred to return to the original state, whether through repair or replacement. For example, insurance policies which replace

damaged items with new purchases tend to produce a loss value which exceeds the damage value. Loss is a more realistic indication of the ultimate, direct economic impact of a disaster.

In order to better delineate the boundary between non-failure and failure damage states with respect to residences, a damage scale was developed for floods (Table 1.4.1). Each damage scale (DS) level represents a threshold in which the hazard event affects the residence in a fundamentally different, and more damaging, manner. This scale defines a boundary between non-failure (often with damage) and failure (obviously with damage):

- DS0 represents non-damage (and thus non-failure).
- DS1 and DS2 represent damage but not failure.
- DS3 represents the transition from non-failure to failure in that a residence component fails.
- DS4 and DS5 represent structural failure.

Table 1.4.1: Damage Scale for Floods

Damage Scale (DS) Level	Water Interaction with Structure
DS0	No water contact with structure.
DS1	Water contacts outside of structure but does not enter.
DS2	Water infiltrates (i.e. seeps or leaks in through small apertures). OR External features are damaged or removed by water or debris.
DS3	Water or debris penetrates through a closed or covered opening (probably by breaking the opening); for example, a window or a door.
DS4	Water or debris penetrates through a route not including an opening (structural integrity is attacked); for example, a wall or roof.
DS5	Structure is damaged beyond repair; for example, walls collapse, the structure moves, or the foundation is undermined.

The advantages of the categories in Table 1.4.1 is that they are relatively distinct and identifiable, but ambiguities may arise. For example, a flood with high velocity flowing smoothly could lift a caravan off its foundations and deposit it safely in another location without further damage. The structure has been damaged, with respect to broken securing devices and a change of position, but classification of failure or non-failure is less lucid. The caravan may be intact and usable yet returning it to its original state may be costly and difficult. Another example occurs if the cleaning and repair of a flooded but intact and usable house would cost more than replacement. Damage has occurred but the occurrence of failure is unclear. Likewise, at DS0, humidity from nearby, long-standing flood water could affect residence contents, especially electrical devices (after Crichton, 2002). Despite these rare

examples of ambiguities, Table 1.4.1 provides a clear framework for assessing flood damage to residences.

1.5 Summary

The work presented in this dissertation has been described, previewed, and placed in context. The topics mentioned must now be explored to meet the objectives mentioned in Section 1.3.1. The first step is to detail the specific nature of the dissertation's topic: risk from the flood natural hazard and residence vulnerability in coastal, eastern England.

2. Risk Environment for Residences in Coastal, Eastern England

2.1 Introduction

This Chapter describes the development of a method for quantitatively describing the risk environment for residences subjected to floods in coastal, eastern England. The principles and methods described are applicable to risks from other natural hazards.

2.2 Quantitative Risk

2.2.1 Defining Risk

The Oxford English Dictionary (OED, 2002) defines the noun "risk" in an implicitly quantitative manner in definition 2. Definition 2.a states that risk is "The chance or hazard of commercial loss, spec. in the case of insured property or goods". Definition 2.b further specifies this approach through a quotation from 1841 which reads "In the theory of Probabilities the risk of loss or gain means such a fraction of the sum to be lost or gained as expresses the chance of losing or gaining it". This definition correlates well with the modern definition of mathematical expectation (e.g. Råde and Westergren, 1990; Turner, 1970):

$$EXP[f(x_r)] = \sum_{All\ x_r} P[f(x_r)] \times f(x_r) \qquad \text{(for a discrete function)}$$

$$= \int_{All\ x_r} P[f(x_r)] \times f(x_r)\, dx_r \qquad \text{(for a continuous function)}$$

In prose, this expectation equation states that mathematical expectation is the expected gain or loss over a specified timeframe. Expectation equals the product of the outcome and the probability of achieving that outcome during a specified timeframe, summed over all possible outcomes. Although OED (2002) fails to explicitly incorporate the "specified timeframe" component into their risk definition, probability by definition always incorporates a time component. This time component tends to be expressed with standard temporal units such as seconds or years for continuous $f(x_r)$ and with the temporal units of "events" for discrete $f(x_r)$. An example of the latter occurs in calculating the probability of obtaining a certain number of tails for a coin tossed a pre-determined number of times. The coin tosses are the "events".

This definition of risk as an analogue to mathematical expectation, i.e. the anticipated outcome of a probabilistic function over a specified time period, is adopted for this study. The risk definition still requires a meaningful interpretation in the context of residence vulnerability to floods because the expectation equation is generic. $f(x_r)$ may be any real function on any random variable.

Previous natural hazards studies assist in developing an appropriate interpretation:

- Alexander (1991):

  Total risk = Impact of hazard × Elements at risk × Vulnerability of elements at risk

- Blong (1996):

  Risk = Hazard × Vulnerability

- Crichton (1999):

  "'Risk' is the probability of a loss, and this depends on three elements, hazard, vulnerability and exposure". If any of these three elements in risk increases or decreases, then risk increases or decreases respectively.

- Granger *et al.* (1999):

  "Risk (i.e. 'total risk') means the expected number of lives lost, persons injured, damage to property and disruption of economic activity due to a particular natural phenomenon…Total risk can be expressed in pseudo-mathematical form as:

  $$Risk_{(total)} = Hazard \times Elements\ at\ Risk \times Vulnerability$$

  This approach is not only elegant, it is also very practical. Given the complexity of urban communities and the degree to which the various elements of risk are interdependent, the 'total risk' approach is considered mandatory. Further, it also lends itself to quantitative, qualitative and composite analytical approaches"

- Helm (1996):

  Risk = Probability × Consequences, although "this simple product is not sufficient in itself to fully describe the real risk, but...it provides an adequate basis for comparing risks or making resource decisions".

- Smith (1996):

  "Risk is the actual exposure of something of human value to a hazard and is often regarded as the combination of probability and loss".

- UN DHA (1992)

  Risk is "Expected losses (of lives, persons injured, property damaged, and economic activity disrupted) due to a particular hazard for a given area and reference period. Based

on mathematical calculations, risk is the product of hazard and vulnerability". Hazard is "A threatening event, or the probability of occurrence of a potentially damaging phenomenon within a given time period and area". Vulnerability is "Degree of loss (from 0% to 100%) resulting from a potentially damaging phenomenon".

These references indicate that risk is fundamentally a combination of hazard and vulnerability. In order to mathematically combine hazard and vulnerability to quantify risk as mathematical expectation, quantitative descriptions of hazard and vulnerability are necessary.

2.2.2 Defining Hazard and Vulnerability

Natural hazard refers to a characteristic of or phenomenon from the natural environment which has the potential for causing damage to society. Society includes residences. Examples of natural hazards are wind, tornadoes, flood water, soil in floods, rain, hail, tigers, kiwi birds, rickettsia, cosmic rays, asteroids, earthquakes, lava, rocks, and tsunami. This dissertation focuses on flood hazards.

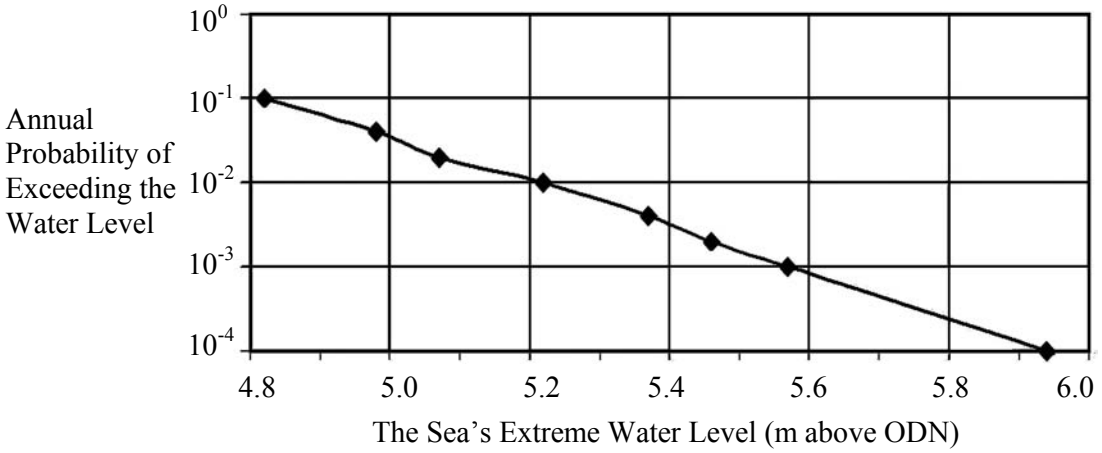
Flood hazard is often quantified by selecting a measurable parameter, such as depth, velocity, or maximum flow rate. Probabilities of exceeding a specific value of this hazard parameter in any given timeframe are determined through observation, experimentation, modelling, or a combination. Figure 2.2.2.1 illustrates flood hazard quantification by calculating probabilities of the sea's extreme water level exceeding a certain value at the mouth of the Humber Estuary.

Vulnerability refers to a characteristic of society which indicates the potential for damage to occur as a result of hazards. Vulnerability (after Capobianco *et al.*, 1999 and Klein and Nicholls, 1999) is a function of:

- resistance, the ability to withstand change due to a hazard;
- resilience, the ability to return to the original state following a hazard event;
- and susceptibility, the current physical state, without taking into account temporal changes.

Capobianco *et al.* (1999) and Klein and Nicholls (1999) define vulnerability for only the natural environment. Their definition is pertinent to both residences and other aspects of society. For example, in considering the property insurance industry, susceptibility may be interpreted as exposure, the accumulated value and proximity to hazard of insured residences (after Crichton, 1999).

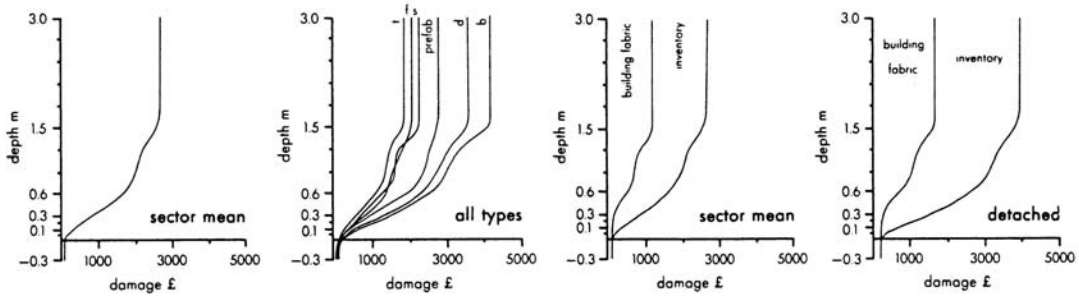
Figure 2.2.2.1: Quantifying Flood Hazard at the Mouth of the Humber Estuary
(data from Dixon and Tawn, 1997)



An example of a house in a flood illustrates this definition's applicability. A house has resistance in its ability to prevent structural collapse due to external water pressure. A house also has resilience in its material's physical properties relating to the ability to dry without damage following inundation. Finally, a house has susceptibility or exposure due to its value and location. The house's physical vulnerability to a flood is influenced by other aspects of society, including social vulnerability such as the occupiers' actions before the flood or the government's long-term economic and development policies.

Residence vulnerability is often quantified by selecting a measurable parameter, such as insured loss or UN DHA's (1992) vulnerability definition. The values of this vulnerability parameter which result from experiencing different hazard parameters are determined, through observation, experimentation, modelling, or a combination. Figure 2.2.2.2 illustrates quantification of U.K. residence flood damage according to flood depth by developing a model incorporating different residence and occupant characteristics (further description of this study appears in Section 2.4.2).

Figure 2.2.2.2: Examples of Flood Depth vs U.K. Residence Damage Curves
(from Penning-Rowsell and Chatterton, 1977)



2.2.3 Risk as a Loss Function

The risk definitions provided in Section 2.2.1 illustrate that some form of multiplying quantitative hazard with quantitative vulnerability to define risk is consistent with the definition of risk as mathematical expectation. Caution is warranted because, paraphrasing Helm (1996), this product does not describe the totality of the real risk but is appropriate as a basis for comparing risks and for risk management decision-making. Lewis (1999) warns that "focus on risk of a given magnitude may cloud our perception of a reality which might in fact be lesser or partial". Similarly, Sayers *et al.* (2002) point out that two calculated risks with equal values may not necessarily be equivalent because a risk dominated by a high hazard probability has different management needs than a risk dominated by high vulnerability. With these provisos in mind, a quantitative definition of risk suitable for this study's focus on residence vulnerability to floods may be sought.

UN DHA (1992) supported by analogies with other authors' definitions in Section 2.2.1 provides a clear, adaptable definition. A straightforward, generalised equation for the definition's core is:

risk to one residence for one hazard event defined by one hazard parameter value

= (probability of the hazard parameter value being equalled or exceeded at the residence within a specified timeframe) × (physical vulnerability of the residence to that hazard parameter value)

This equation resembles the core of the expectation equation in Section 2.2.1:

- risk corresponds to $EXP[f(x_r)]$.
- hazard parameter value exceedance probability corresponds to $P[f(x_r)]$.
- vulnerability to that hazard parameter value corresponds to $f(x_r)$.

This calculation of risk = (hazard parameter value exceedance probability) × (vulnerability to that hazard parameter value) must be summed over all possibilities within the bounds set by the problem. In this study, the bounds are that the hazard is a flood hazard parameter and the elements deemed to be physically vulnerable to the flood hazard parameter are residences. Section 2.5 extends the discussion to multiple flood hazard parameters simultaneously.

Residences are discrete but flood hazard parameters are continuous yielding the risk equation:

$$\text{risk} = \sum_{\text{All residences}} \int_{\text{All values of the hazard parameter}} [\text{(hazard parameter value exceedance probability)} \times \text{(vulnerability to that hazard parameter value)}] \; \text{integrated with respect to the hazard parameter.}$$

This definition of risk is adopted for this dissertation in principle. Deviations from this principle in practice, and the reasons for changes, will be noted, especially in Chapter 13. In particular, Brown's (2003) work ended up considering only specific flood hazard events, each with an occurrence probability of one. Hazard exceedance probabilities were not calculated.

As well, a specific definition of vulnerability in terms of UN DHA's (1992) "degree of loss" is not decided in this dissertation. The core research of this dissertation, and the main contribution to knowledge, is a new system of vulnerability profiling for floods through a physical description of the vulnerability. In order to meet the objectives of this dissertation (Section 1.3.1), this system must be used to calculate risk. When this calculation step is undertaken (Chapter 13), different possibilities for calculating vulnerability, along with consequences of certain choices, are explored.

Using the UN DHA (1992) definitions, the reported risk is dimensionless but is defined on a monetary basis. Assigning monetary values to residences is not as fraught with difficulties as assigning monetary values to invaluable elements such as life, health, culture, and ecosystems or to intangible elements such as aesthetics and emotions. Residences hold invaluable and intangible qualities, but analysing the monetary values of residences tends to be more substantial and less controversial than for other possible losses. Therefore, the risk definition suggested here is pertinent to the residences examined here, but extrapolation beyond residences may not be feasible.

The difference between the disaster mitigation point of view and the property insurance company point of view illustrates this issue. The goal of disaster mitigation tends to be to reduce casualties while the goal of property insurance companies tends to be to reduce property losses. Protecting property often protects occupants, but design strategies for natural disasters exist which sacrifice the structure in order to protect the occupants (e.g. Harris *et al.*, 1992). If a conflict arises between these two objectives when selecting a flood management strategy or when assessing vulnerability, the preferred point of view should be identified.

The specific flood hazards and physical vulnerabilities to explore, and to quantify, with respect to residences in coastal, eastern England must now be determined.

2.3 Flood Hazards in Coastal, Eastern England

Examples of hazard parameters which could be measured to quantify direct flood hazards in coastal, eastern England are:

- Volume.
- Depth.
- Volumetric flow rate.
- Velocity: average and "gusts" (measures of spatiotemporal variation).
- Temperature.
- Salinity.
- Density.
- Percent not water by mass or by volume (e.g. debris and contaminants including silt, stones, sewage, oil, paint, and collapsed residence material) along with specific characteristics of non-water parts such as size, mass, density, velocity, miscibility with water, corrosiveness, and flammability. Without a substantial portion of water in the flood, the hazard would labelled with another term, such as lahar, debris flow, mudflow, landslide, avalanche, or rockfall.
- Wave height, length, and frequency.

Each parameter is a function of three-dimensional space (x,y,z) and time (t).

Although this study focuses on direct flood hazards, the wider context can rarely be ignored. Examples of water hazards in combination with other hazards are:

- Wind-driven rain.
- Wind and low atmospheric pressure augmenting the tide level to produce a storm surge and large waves.
- Wind causing waves during a storm after flooding has turned a residential area into a "lake".
- Biota's range expanding due to the flood or flood-induced dampness. For example, jellyfish or whales swimming through a residential area or fungus or algae growing on a residence wall.

- Rainfall- or water flow- induced landslides.
- Storm- and wave-induced coastal erosion.

A linear sequence may occur in which one hazard event causes a structural weakness meaning that the residence cannot act at the design level for the next hazard event. Time may not be available for maintenance or repair work between the two events.

To determine the effect of a hazard on a residence, the hazard parameter must be translated into a form which indicates an effect on the residence. For example, forces, pressures, and energy transfers may be calculated from flood depth and velocity characteristics. Collectively, such impacts on residences due to hazard parameters are termed "actions" and are described in Chapter 3. Chapter 3 also scopes the flood actions to be examined in detail in this dissertation.

In interpreting and recording the hazard parameters, the presence of the residence may be important. For example, flood velocity would change direction, possibly with augmented vertical components, upon encountering a residence. Flood flow parallel to non-failing residence yields a boundary layer in the fluid, altering the velocity profile. Furthermore, as the flood water flows around the residence's corners, eddies are formed, substantially altering the fluid's velocity and flux profiles.

2.4 Residence Physical Vulnerability to Flood Hazards in Coastal, Eastern England

2.4.1 Introduction

Damage and losses from floods are categorised as either direct, resulting from the physical contact of flood water with damageable property, or indirect, resulting from the interruption or disruption of social and economic activities. Damage and losses from floods are also either tangible, for which a monetary value can be easily assigned, or intangible, for which a monetary value cannot be easily assigned. Examples of damage in these four categories are:

- direct, tangible: food, appliances, residence structural collapse.
- direct, intangible: photographs and negatives, heirlooms, an archaeological site, drownings.
- indirect, tangible: days absent from work, changed spending patterns.
- indirect, intangible: quality of life lessened due to stress, delays in formal education.

In focusing on physical damage to residences, this study considers direct, tangible damage. Assigning a monetary value is relatively easy when choosing a method for quantifying vulnerability, as implied in Section 2.2.3.

2.4.2 Past Studies

Examples of non-U.K. studies which estimate and analyse flood damage and losses to residential properties—and at times other aspects of society—are listed in Table 2.4.2.1. Some studies analyse flood damage or losses from events, some studies estimate flood damage or losses for future events, and some studies use observed damage or losses to estimate future damage. U.K. studies are discussed after Table 2.4.2.1 because detailing U.K. research is helpful for understanding the residences and vulnerabilities in this study's focus area of coastal, eastern England.

Table 2.4.2.1: Examples of Studies of Residence Flood Damage and Losses Outside the U.K.

References	Flood Type	Geographic Area	Hazard Considered	Measure of Residence Vulnerability
Appelbaum (1985)	Fresh water.	Baltimore District, U.S.A.	Depth.	Damage in monetary terms and as a percentage of replacement value.
Beck *et al.* (2002)	River.	Luxembourg.	Depth and velocity.	Potential damage noted as a degree of acceptability.
Black (1975)	Fresh water.	U.S.A.	Depth and velocity.	Structure moves or does not move.
CH2M Hill (1974)	River.	Willamette Valley, Oregon, U.S.A.	Depth and velocity.	For depth, damage in monetary value and in percentage of total value. For depth and velocity, collapse or no collapse.
"Child of ANUFLOOD" (1998), Smith (1991), and Zerger (2000)	Fresh water.	Australia.	Depth with velocity as an optional input.	Damage in monetary terms. Building failure is predicted from Black (1975).
Davis and Skaggs (1992)	River.	U.S.A.	Depth.	Damage in monetary terms and as percentage of value.
DeGagne (1999)	River.	Manitoba, Canada.	Depth.	Damage in monetary terms and as percentage of assessed value.
Green and Parker (1994)	Fresh water.	Australia, Germany, Japan, and the U.S.A.	Depth. The Japanese studies also consider sediment load.	The authors compare depth-damage curves, concluding that the shape of the curves is reasonably similar for depths of less than 1.2 m.
Hubert *et al.* (c. 1996)	Urban runoff.	France.	Depth and duration.	Damage in monetary terms.
Islam (1997)	River, tidal, and "flash flood".	Bangladesh.	River: Depth and duration. Flash: Velocity. Tidal: Velocity and salinity.	Damage in monetary terms and percentage of total value. Some social variables were considered.
Kato and Torii (2002)	Storm surge.	Japan.	Depth, sediment depth, and duration.	Damage in percentage of total value.

Reese and Markau (2002)	Storm surge.	Schleswig-Holstein, Germany	Depth.	Damage in percentage of total value.
Risk Frontiers (2002)	Fresh water.	Australia.	Depth.	Damage and losses are not considered.
Sangrey *et al.* (1975)	River.	Elmira, New York, U.S.A.	Depth and velocity.	Structure destroyed or survived.
Smith and Greenaway (1980)	River.	Lismore, New South Wales, Australia	Depth.	Damage in monetary terms.
Smith and Greenaway (1994)	Storm surge with consideration to conjunctive river-surge flooding.	Mackay, Queensland, Australia.	Depth, velocity, and wave height.	Damage in monetary terms.
Smith *et al.* (1981)	River.	South Africa	Depth.	Damage in monetary terms.
Torterotot *et al.* (1992)	River.	France.	Depth and duration.	Damage in monetary terms. Warning type, action taken, and building and occupancy characteristics were also considered.
USACE (1993)	River.	U.S.A.	Depth.	Damage in monetary terms and percentage of total value.
USACE (2000)	Fresh water.	U.S.A.	Depth.	Damage in percent of value.

Additionally, Smith (1994) and USACE (1996) write overviews on developing depth-damage functions. Smith (1994) notes that velocity may be a concern in rare instances and provides building failure data from Black (1975). USACE (1996) suggests that velocity, duration, sediment, frequency, flood warning, and building characteristics are factors which influence flood damage too.

In the U.K., the Flood Hazard Research Centre (FHRC) at Middlesex University, London has completed studies estimating the vulnerability of U.K. residences to floods in monetary terms. FHRC calculated curves of monetary damage as a function of slow-rise flood depth. Their first major publication (Penning-Rowsell and Chatterton, 1977) systemised the assessment of the benefits of flood alleviation for both urban areas and agricultural land using synthesised data for direct flood water damage. Nine sectors, categorised as land use types, were examined including residential properties.

The residential sector was subdivided into a matrix of:

- five housing types (detached, semi-detached, terrace, bungalow, and flat);
- four housing ages (pre-1918, 1918-1938, 1939-1965, and post-1965);
- four social categories reduced from six social classes; and
- the independent housing type category of "pre-fab" which is subdivided by neither age nor social class.

Depth-damage curves were developed for:

- no flood warning along with flood warnings of 0.5 h, 2 h, and 4 h; and
- two arbitrary flood durations, short which is less than 12 hours and long which is more than 12 hours.

The curves for residences are reported for five depths above floor level and one depth below floor level. Smooth interpolation between the depth values is assumed for all curves.

Using this work as the main basis, subsequent reports update the various land use and residential categories while refining the methods in order to produce software packages for desktop computers. The next major FHRC work (Parker *et al.*, 1987) used synthesised data to estimate flood protection benefits for urban areas, including indirect benefits, again on a depth-damage basis. Subsequently, Suleman *et al.* (1988) incorporated clean-up costs into the depth-damage calculations and noted that building fabric damage accounts for more than half of total flood damage.

N'Jai *et al.* (1990) formed part of these continued pricing and assessment updates from FHRC. N'Jai *et al.* (1990) present further revisions to the land use classifications for calculating depth-damage curves. Each of the five residential building types was now described with one of seven ages: pre-1918, inter-war (1918-1938), post-war (1939-65), modern (post-1965), post-1975 standard, post-1975 timber-framed, and post-1975 garage/utility. The category "Mobile homes (caravans) pre-fab" was revised based on results from FHRC (1983). FHRC (1983) described the flood depths for which a caravan would float and the chance of destruction of a floating caravan. The most recent FHRC manual is Penning-Rowsell *et al.* (1992) which lists percentages by which to increase the flood damage values in the case of a salt water flood. The assumption is a simple contrast between salt water flooding and fresh water flooding.

The work of N'Jai *et al.* (1990) was updated by FHRC during the 1990s and was eventually redeveloped by Experian (2000) who updated the values to October 1998. Experian's (2000) most prominent change was to provide the depth-damage values for lifestyle, rather than residence, classes. Separate tables are provided for depth-damage curves for each of the two flood durations (less than and more than 12 hours) and for each of 52 lifestyle types clustered in 12 lifestyle groups. These lifestyle classes were built using 86 demographic and housing variables for Great Britain. Demographic variables include the occupants' ethnicity, age, employment sector, and principal transport mode. Housing variables include financial status (e.g. rent, own outright, or mortgaged), rooms per person, presence or absence of central heating, and WC type (e.g. shared or not inside).

Penning-Rowsell *et al.* (1992) briefly mention velocity's impact on flood damage, based on research by Clausen (1989). Clausen (1989) concludes, based on empirical data from mainly the Dale Dyke dam failure in Sheffield on 11 March 1864, that:

- $v < 2$ m/s or $f \times v < 3$ m^2/s yields only "inundation damage";
- $v > 2$ m/s and 3 m^2/s $< f \times v < 7$ m^2/s yields "partial damage"; and
- $v > 2$ m/s and $f \times v > 7$ m^2/s yields "total destruction".

These original results from Clausen (1989) are not exactly the same as reported in Penning-Rowsell *et al.* (1992).

Clausen (2000) mentioned that she conducted an extensive, international search for prior studies of this nature and found "very little real information". The few studies uncovered were relevant mostly to American building types uncommon in the U.K. Clausen (1989)

identifies the main hazard parameters of interest for estimating flood damage as water depth, flow velocity, bed shear stress, dynamic forces (flow momentum, stream power, depth times speed), rate of flood rise, and debris potential of the landscape. She states that flood duration is a significant factor, but does not include it as a separate parameter.

Clausen (1989) chooses $f \times v$ as a parameter due to its simplicity and to emulate other researchers, although she notes that little theoretical justification exists for its use. The other main weakness which she identifies is that f refers to point depth in the flood whereas v represents the average speed in the direction of flow. Her concern was that the Dale Dyke dam failure occurred in a valley, hence water speeds across the valley in the direction of flow would likely have varied significantly. Average speed in the direction of flow therefore correlates poorly with observed damage, since higher speeds and thus more damage would be expected towards the valley's longitudinal centre. Clausen (1989) states that her conclusions "are obviously not definitive, but they do provide a first approximation to use in the absence of more reliable results".

Black and Evans (1999) undertook an empirical exercise which analysed insurance claims from seven floods in the U.K. during the 1990s for which a loss adjuster was needed. These claims had to be above a threshold which varied depending on the event and the loss adjuster, but which was usually approximately UK£5,000 at the time of the flood. To characterise the floods, depth, duration, velocity, contaminating substances, salinity, and season (summer or winter) were considered. Comprehensive data on the flood events were available for only depth and season. Residences were classified as semi-detached, detached, ground-floor flat, terraced, and bungalow. Losses, which covered any alternative accommodation needed for occupants of flooded properties, were reported in monetary terms as a function of total sum insured in monetary terms.

Commercial firms develop and apply flood loss models for U.K. residences. Muir-Wood (1999) describes the East Coast UK Storm Surge Flood model developed by Risk Management Solutions, Inc. Depth and duration are considered for calculating losses along with "A velocity damage modifier for depths in excess of 1.5 m, declining exponentially with distance from the source of flood water" (Muir-Wood, 1999). Losses are reported in monetary terms and as a percentage of the total sum insured.

Risk Management Solutions, Inc.'s U.K. River Flood Model (RMS, 2001) covers England, Scotland, and Wales. The model includes "separate damage relationships for river inundation and for direct rain flooding…Inundation vulnerability functions relate damage to flood depth, and direct rainfall vulnerability functions relate damage to rainfall intensity". Despite the use of the term "damage", the output is for losses according to this study's definitions. Losses are reported in monetary terms and as a percentage of the total sum insured.

Toothill (2002) describes a European flood model developed by EQECAT, a division of ABS Consulting, which includes a probabilistic storm surge model for coastal, eastern England between the Humber Estuary area and the Thames Estuary. This model was being revised and updated in August 2002. The only hazard parameter considered is slow-rise flood depth and losses are provided in monetary terms for the area affected, which can be adjusted to indicate insurance losses.

2.4.3 Past Studies: Overall Analysis

The most detailed U.K. studies on damage and loss for residences in floods have considered almost exclusively depth-damage curves. The flood depth is assumed to rise slowly so that damage occurs only due to water touching the damaged item or structure and not due to any physical force, pressure, or energy imparted to an item, component, or structure. DS1 and DS2 in Table 1.4.1 are explored without considering the higher DS levels.

Duration is specifically considered in many U.K. studies, but justification is not provided for the duration times considered. Penning-Rowsell (1981) states, without supporting evidence, that:

> The part played by flood water velocities in producing damage is assumed to be small except in rare cases of structural failure. The more minor effects of velocity are generally measured by the depth variable.

He further suggests that duration is as unimportant, although sediment loading "affects clean-up costs" and sewage contamination "has been assumed to affect damage values" (Penning-Rowsell, 1981). Meanwhile, Green and Parker (1994) write "Assuming that damage data is standardised for depth then, for direct damages, velocity and sediment loads appear to [be] the next most critical determinants".

In contrast, many non-U.K. studies listed in Table 2.4.2.1 indicate the importance of flood characteristics other than depth. As a further example, USACE (1998) describes the impact in 1993 of velocity-induced hydrodynamic pressures when a Nutwood, Illinois river levee overtopped without breaching. Two residences collapsed and two residences partially collapsed due to failure of concrete foundation walls built to raise the residences above potential flood levels. USACE (1998) proposes a flood proofing matrix delineating thresholds believed to be important for different damage scenarios:

f_{diff}:	shallow (< 0.9 m), moderate (0.9 m to 1.8 m), or deep (> 1.8 m)
v:	slow (< 0.9 m/s), moderate (0.9 m/s to 1.5 m/s), or fast (> 1.5 m/s)
Flash flooding:	yes (less than 1 hour) or no
Ice and debris:	yes or no
Site Location:	coastal or riverine
Soil Type:	permeable or impermeable

Three sets of structural characteristics are also provided.

Justification for the categories is not provided, but USACE (1997) notes that most buildings would collapse for $f_{diff} > 0.9$ m. USACE (1998) further suggests that for $f_{diff} > 0.9$ m, the structure would need to be designed to resist both hydrostatic and buoyancy forces. This $f_{diff} = 0.9$ m threshold may come from experimental results in USACE (1988) (details of the experiments are in Chapter 9).

USACE (1995) describes water loads (hydrostatic and hydrodynamic), debris impact loads, soil loads, wave loads, and uplift pressures as factors in structural flood damage. With respect to water loads, USACE (1995) suggests that only hydrostatic loads must be considered in designing for $v < 1.5$ m/s, while flood proofing measures would be economically ineffective for $v > 3.0$ m/s. For $v < 3.0$ m/s, "dynamic effects of the moving water may be converted into equivalent hydrostatic loads" with the excess depth proportional to v^2 (USACE, 1995).

Despite such statements on the importance of non-depth flood characteristics, they are not investigated as thoroughly and systematically as slow-rise depth. Some decisions are arbitrary, such as the dominance of the $f \times v$ variable (e.g. Clausen, 1989; Sangrey, 1975). A physical meaning for this variable is not articulated, although it may relate to momentum = mass $\times$ v = $\rho_w \times$ volume $\times$ v = $\rho_w \times$ flood horizontal area $\times$ f $\times$ v. ρ_w is constant while flood horizontal area is perhaps considered constant leaving $f \times v$ as the variable.

This critique of physical meanings not being articulated generally applies to the approach traditionally taken for flood damage research. An understanding of the physical mechanisms which result in flood damage to residences is not discussed. These studies' flood damage functions are frequently derived from synthesised and/or collected damage, loss, and pricing data, thereby yielding only a description of the damage which would occur given a flood of certain characteristics. A physical understanding of when, how, and why structural failure would occur is not significantly present. For the U.K., the potential impact on flood losses of the structural failure is rarely discussed. The traditional approach is needed and is appropriate for many uses, but a gap remains to be filled in the understanding of physical processes which are occurring.

In particular, damage and losses beyond slow-rise flood depth need to be investigated in detail. Such damage may result from energy transfer, forces, or pressures leading to effects on residences including structural failure, doors being forced open, glass breaking, roofs collapsing, or foundations being undermined. DS3, DS4, and DS5 in Table 1.4.1 represent the current gap in knowledge with respect to damage and losses to residences from floods, especially in the U.K.

2.4.4 Conclusions

The nature of physical vulnerability of residences to floods has been described and past studies on the topic have been examined. These studies provide useful results on an important aspect of vulnerability, but leave a gap in the understanding of vulnerability with respect to the most potentially damaging aspects of floods. As well, an explanation of how damage, either with or without failure, occurs to residences in floods is rarely provided. This dissertation can make an original contribution to research by tackling this knowledge gap.

2.5 Developing the Risk Environment

Section 2.3 established an understanding of hazard for this study. Section 2.4 established an understanding of vulnerability for this study. The risk environment for this study can now be completely defined, based on the definition in Section 2.2.

From Section 2.2.3, the risk equation is:

$$\text{risk} = \sum_{\text{All residences}} \int_{\text{All values of the hazard parameter}} [\text{(hazard parameter value exceedance probability)} \times \text{(vulnerability to that hazard parameter value)}] \text{ integrated with respect to the hazard parameter.}$$

This equation may be interpreted for this study in terms of the knowledge gap identified in Section 2.4. The hazard exceedance probability would be for flood hazard parameters other than slow-rise depth. Vulnerability would examine the mechanisms of failure of residences, would predict which failures would occur for which residences, and would estimate the proportion of the total value of each residence lost due to being subjected to the hazard. In theory, the risk would measure the expected proportion of the total value of residences in a community which would be expected to be lost over a given timeframe.

To appropriately describe a risk environment in practice, limitations of the risk equation must be considered. To determine a hazard exceedance probability, specific hazard parameters must be selected. The probability density function would be multi-dimensional so that exceedance probabilities would represent a specific set of hazard parameters; for example, the probability that $f \geq 2.3$ m, $f_{diff\,max} \geq 2.0$ m, and $v \geq 1.2$ m/s.

This set of hazard parameters would also determine the damage which the residence experiences, representing the vulnerability, through understanding the processes by which residences fail in certain hazard scenarios. As noted in Section 2.4, this approach is not covered in the literature. This dissertation aims to fill this gap at the first order. The risk environment examined in this dissertation for residences in coastal, eastern England is plugging the knowledge gap of non-f damage potential at the first order.

This first order approach assists in identifying the main hazard parameters to examine. The choice of hazard parameters emerges from the potential damage which could be caused and the spatial scale over which these parameters could be examined. Chapter 3 categorises flood hazard characteristics as "flood actions on a residence" and identifies those to be examined in this dissertation.

Similarly, the first order approach assists in identifying the main damage modes to examine for calculating vulnerability. The choice of damage modes emerges from those damage

modes which would be normally expected under flood scenarios based on the identified flood hazard parameters.

Chapter 7 starts the process of examining the consequences of flood actions on residences. After specific analyses in Chapters 8, 9, and 10, Chapter 11 completes the process. Chapter 12 then combines the results into matrices which qualitatively describe vulnerability. These vulnerability matrices are used in Chapter 13 for a quantitative assessment of vulnerability as required in the risk equation.

An essential input to this process is characterising the residences which exist in the areas examined. Chapters 4, 5, and 6 establish the residences which exist and their features which are vulnerable to the flood hazard characteristics selected. This survey process is undertaken in parallel with the analysis of the consequences of flood actions on residences (Figure 1.3.2.1).

Conducting the survey and vulnerability analysis concurrently rather than independently yields continual feedback and exchange of information between them. Therefore, the originality in developing an understanding of residence vulnerability to flood hazards is enhanced. The theoretical understanding of the processes which induce damage is combined with a clear understanding of the residence characteristics which exist in flood-vulnerable locales. The risk environment developed reflects the original contribution made to this field.

3. Flood Actions on Residences

3.1 Introduction

As noted in Section 2.4 analyses of flood damage to residences in the U.K. traditionally focused on damage from slow-rise flood depth. Water depth in slow-rise floods tended to be the flood characteristic most frequently analysed in detail. This study's mandate is to investigate flood characteristics not previously examined in detail in order to contribute new knowledge and techniques.

Such characteristics include forces, pressures, chemical reactions, and other impacts which a flood could impose on a residence. Collectively, they are termed "actions" for this study and refer to something to which a structure responds. Flood actions describe acts which a flood could do to a residence, potentially causing damage and failure. Full analysis of flood actions will permit losses from potential flood events to be calculated more comprehensively. This Chapter defines and quantifies flood actions on residences and suggests a hierarchy for selecting those actions which should be examined in more detail.

3.2 Overview of Flood Actions

3.2.1 Introduction

Flood actions are identified and classified in this Section based on the discussion and references in Section 2.4 along with further material noted here. This Section provides an overview of qualitative and quantitative characteristics of flood actions. The material is illustrative, not comprehensive.

3.2.2 Hydrostatic Actions

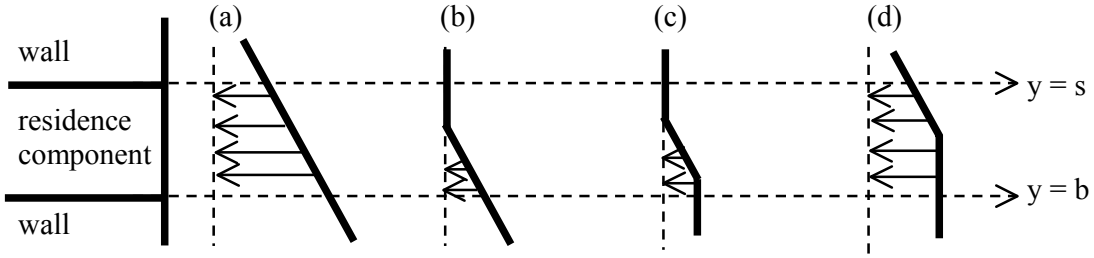
Two forms of hydrostatic action exist: lateral pressure and capillary rise.

The lateral pressure imparted by a depth of water against a residence or residence's component is:

$\Delta P = \rho_w g(f_{diff} - y) = \Delta P_{hydrostatic\ at\ y=0} - \rho_w gy$ for $b \le y \le f_{diff}$

$\Delta P = 0$ for $y > f_{diff}$

Figure 3.2.2.1: Pressure Distribution On Residence Component In Each Situation



Depending on the specific flood situation and residence component being considered, this basic equation has four variations (Figure 3.2.2.1):

(a) Water covers the entire residence component on one side yielding a linear pressure over the entire residence component.

(b) Water rises partway up the residence component on one side.

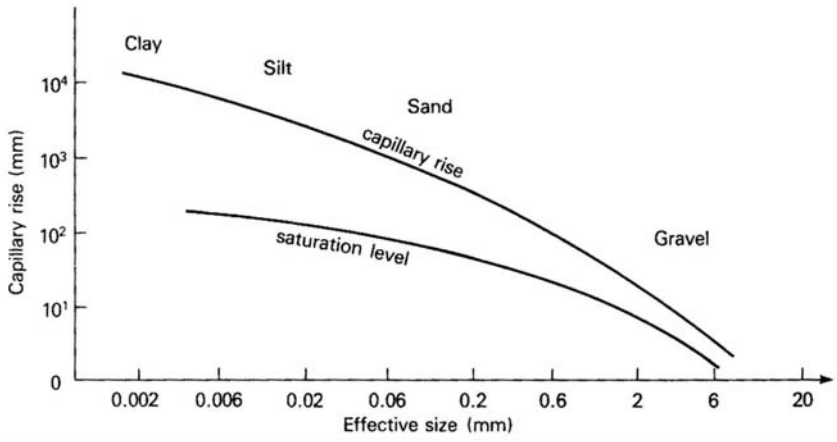
(c) Water rises partway up the residence component on both sides, but to different y-values on each side.

(d) Water entirely covers the residence component on one side and rises partway up the other side.

Whether the water level is greater on the outside or inside, the same ΔP is imparted to the residence component—which may be the wall itself. If ΔP were from the inside and were relatively small, a strong wind could conceivably prevent failure by balancing the pressure. Localised wind pressure is highly variable meaning that in most such cases, the residence component would fail at some point anyway.

Capillary rise inside a residence's components which water has contacted would cause damage at a height above f or that reached by waves. Hoffmann and Niesel (1995) write "capillary effects occur in pores between about 0.1 and 100 μm diameter" and they indicate that the pore sizes of masonry units and render fall within that range. They note that "accessible pore volume and portion of capillary-active pore sizes" are important parameters to consider in determining height of capillary rise through a material. For a brickwork wall, Hoffmann and Niesel (1995) provide a series of equations requiring several empirical parameters for calculating maximum rise height, but no such calculations are carried out. Huelman and Corrin (1997) suggest 0.45 m as an approximate upper limit for capillary rise following a flood, depending on the residence's materials.

Figure 3.2.2.2: Capillary Rise in Soil (modified from Whitlow, 1983)



Capillary rise may also occur in soil, above the water table. Residences which encounter capillary water may absorb it resulting in damage. As illustrated in Figure 3.2.2.2 modified from Whitlow (1983), soil saturated with capillary water may occur up to 0.5 m above the water table. Partial saturation with capillary water may occur more than 10 m above the water table for fine soils such as clay.

3.2.3 Hydrodynamic Actions

Five forms of hydrodynamic actions exist: three from velocity (including turbulence) and two from waves.

The lateral pressure imparted by water flowing around a residence may be taken as $\Delta P = \frac{1}{2}\rho v^2$ for a first-order approximation. This value represents the dynamic pressure due to steady flow of an inviscid, incompressible fluid with negligible heat transfer and shaft work. This pressure occurs at the stagnation point of a fluid flowing around a bluff body, but may be used for ΔP over a residence wall or component as a first-order approximation.

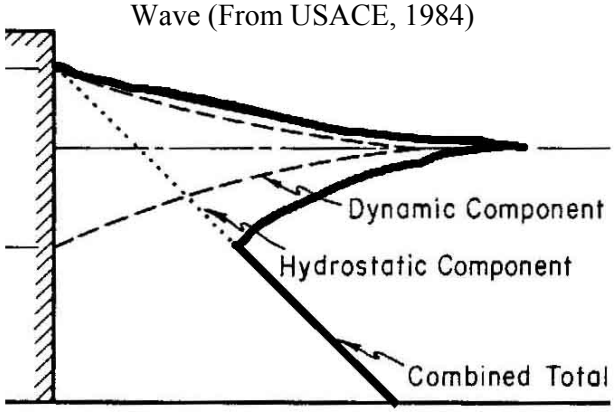
Localised changes in v, and therefore ΔP, occur when water flows around corners of a residence or through gaps. v tends to increase, creating suction forces and producing higher impacts due to local v. A residence which withstands the $\frac{1}{2}\rho v^2$ direct pressure may succumb to the local variations or may be affected by resulting erosion (Section 3.2.4).

Turbulence is irregular fluctuations in v, in either or both the magnitude and the direction. Eddies, vortices, surface choppiness, gusts, and rapid but short-lived changes in f or f_{diff}, all

distinct from waves, may result. Turbulence can be highly variable over short spatial and temporal scales making quantitative prediction difficult.

Non-breaking waves' peaks and troughs will respectively increase and decrease the pressures and total force exerted on a residence. Peaks can add a maximum force approximately equivalent to the hydrostatic force while troughs may decrease the total force by up to 40%—beyond these limits, the wave would break (USACE, 1984). The exact change in total force depends on the ratio of wave height to water depth which is less than about 0.70 for non-breaking waves (USACE, 1984). ΔP at any point on the residence component will obviously change in proportion to the depth of water raised or lowered by the wave's peak and trough respectively. The rate of change of forces and pressures depends on the wave's period.

Figure 3.2.3.1: The Pressure Function of a Breaking Wave (From USACE, 1984)



Waves breaking in, over, through, or near a residence can impart large pressures compared to other hydrodynamic actions. Lewis (1983) records that in Chiswell in coastal, southern England, "in December 1978 and in February 1979 waves overtopped the [shingle] bank with such force that several buildings were damaged". For breaking waves, the pressure function is illustrated in Figure 3.2.3.1. The peak dynamic pressures of breaking waves can be "as much as 15 to 18 times those calculated for nonbreaking waves [but these values] should be used with caution and only until a more accurate method of calculation is found" (USACE, 1984). Lloyd and Harper (1984) use USACE's (1984) method to calculate ΔP generated by breaking waves for different scenarios (Table 3.2.3.1). For f = 0.5 m, peak dynamic pressure is just under 50 kPa; for most variations at f = 2.5 m, peak dynamic pressure is above 500 kPa.

Table 3.2.3.1: Pressures Generated by Breaking Waves (From Lloyd and Harper, 1984)

Peak Static Pressure (kPa)	Peak Dynamic Pressure (kPa)	Flood Depth (m)	Wave Height (m)	Environmental Conditions	
				Bed Slope	Exposure
1.9	42.9	0.5	0.64	1:20	sheltered bay
1.9	44.3	0.5	0.50	1:50	ocean front
2.4	44.3	0.5	0.50	1:50	sheltered bay
2.4	45.7	0.5	0.39	0	sheltered bay
2.8	46.0	0.5	0.39	0	ocean front
3.1	51.0	0.5	0.76	1:20	ocean front
3.7	129	1.0	0.93	1:50	sheltered bay
3.8	130	1.0	1.16	1:20	sheltered bay
4.5	134	1.0	0.79	0	sheltered bay
4.7	135	1.0	0.79	0	ocean front
5.7	135	1.0	0.97	1:50	ocean front
5.7	154	1.0	1.38	1:20	ocean front
5.7	238	1.5	1.58	1:20	sheltered bay
6.5	241	1.5	1.33	1:50	sheltered bay
6.7	248	1.5	1.18	0	ocean front
7.1	250	1.5	1.18	0	sheltered bay
7.6	261	1.5	1.44	1:50	ocean front
7.6	296	1.5	1.96	1:20	ocean front
7.8	359	2.0	1.91	1:20	sheltered bay
8.3	371	2.0	1.69	1:50	sheltered bay
9.3	390	2.0	1.56	0	ocean front
9.3	391	2.0	1.56	0	sheltered bay
9.5	415	2.0	1.89	1:50	ocean front
9.5	475	2.0	2.53	1:20	ocean front
9.6	500	2.5	2.24	1:20	sheltered bay
9.8	512	2.5	2.00	1:50	sheltered bay
11	560	2.5	1.96	0	ocean front
11	562	2.5	1.96	0	sheltered bay
12	591	2.5	2.31	1:50	ocean front
15	694	2.5	3.11	1:20	ocean front

3.2.4 Erosion Actions

Moving water may cause erosion by scouring away soil from the sides or bed along which the water flows. Baker (1988), Bull (1988), Carter (1988), Hamill (2001), Komar (1988), Nelson *et al.* (2000), Rooseboom and le Grange (1994), and Whitehouse (1998) describe erosion and scour analysis. Their work is summarised in this paragraph. Two principal phenomena occur: entrainment of sediment in water and horizontal movement of the entrained sediment. The main water parameters involved in such analysis are f, v, and ρ_w, although volumetric flow rate and kinematic viscosity are considered at times. The main sediment property is an

index representing grain diameter but sediment density and grain cross-sectional area may be included. Bed roughness, which changes due to erosion, is a factor too. The erosion mechanisms usually cited are lift and drag forces, but turbulence may produce instantaneous upward forces large enough to cause entrainment. Turbulence underneath waves, such as the orbital velocity, may lead to sediment entrainment and transport too.

Water seeping through soil may physically move the soil. Ubell (1995) and Whitlow (1983) describe the seepage action which occurs as water infiltrates through soil. This force per unit volume is equal to $\rho_w g$ multiplied by the hydraulic gradient. Ubell (1995) writes "The seepage force acting on the soil particles will cause them to move if not opposed by other greater forces acting in the opposite direction". Water may destabilise soil on slopes causing landslips which would destabilise residences or damage them from direct impact.

3.2.5 Buoyancy Action

The buoyancy force is a function of the submerged volume of the object, in this case the residence. This volume equals the volume of water which the residence displaces, i.e. $A \times f$. Thus, the buoyancy force is $\rho_w g A f$. This buoyancy force is an uplift force which could result in the residence, or parts of it, floating. Hydrodynamic actions or the hydrostatic lateral pressure may then displace the floating residence or parts, potentially causing damage, destabilisation, or complete destruction (FHRC, 1983; Black, 1975; Sangrey *et al.*, 1975). Buoyancy could also affect lighter structures such as shacks or sheds, thereby contributing to debris, along with residence components such as pipes or fuel or water storage tanks.

3.2.6 Debris Actions

Three forms of debris actions exist: static actions, dynamic actions, and erosion. Debris refers to solids in the flood, so chemical, nuclear, and biological actions (Section 3.2.7) may be relevant too. At times, the solids may be such a prominent part of a flood wave that the flow is no longer considered to be a water flood. Costa (1988) provides one example of a simple rheologic classification of flows (Table 3.2.6.1). Mainali and Rajaratnam (1991) and Newson (1989) each provide more complicated schemes focused on the higher sediment concentrations. LACOE (1997) illustrates the immense impacts which debris flows may have on residences as well as on lives and landscapes.

Table 3.2.6.1: A Simple Classification Scheme for Flows (From Costa, 1988)

(Assuming silt and clay content < 10%.)

Flow	Sediment Concentration In Entire Flow		Bulk Density (kg/m^3)	Major Sediment-Support Mechanism
	by Weight	By Volume		
Water Flood	1%-40%	0.4%-20%	1,010-1,330	Electrostatic forces, turbulence
Hyperconcentrated	40%-70%	20%-47%	1,330-1,800	Buoyancy, dispersive stress, turbulence
Debris Flow	70%-90%	47%-77%	1,800-2,300	Cohesion, buoyancy, dispersive stress, structural support

Static debris actions would occur due to sediment accumulating externally or internally to a residence. USACE (1984) writes "the forces exerted on a wall by soil backfill depend on the physical characteristics of the soil particles, the degree of soil compaction and saturation, the geometry of the soil mass, the movements of the wall caused by the action of the backfill, and the foundation deformation". In a flood, the soil backfill would be sediment deposited by the flood. USACE (1984) and Thorley (1969) provide equations for the different forces exerted in a form for which the force imposed equals $0.5\rho_{soil}gy^2Ч$. y represents the height of the soil backfill or deposited sediment. Ч is determined from soil properties and scenario geometry such as the verticality of the residence component subject to the forces. USACE (1984) and Thorley (1969) provide data tables for calculating Ч.

Dynamic debris actions would occur when debris moved by water impacts a residence. The impact could be from outside, such as a cow or car. The impact could also be from inside, such as a couch or table floating and hitting the ceiling, an internal wall, or a window. Dynamic debris actions may be either forces or pressures. An example of a force, concentrated and applied with shock, is a log floating in moving water which impacts a residence. Likewise, Lewis (1999) states that during storm surges in Chiswell in southern England, "waves hurl stones and pebbles, causing their own impact damage to roofs and windows". An example of a pressure applied over a relatively long duration is a hyperconcentrated or debris flow flowing around a residence.

Debris could cause severe erosion, such as pebbles and saturated computers being dragged along with the flow and gouging out soil from the sides or bed of the flow channel. The consequences of erosion would be similar to those in Section 3.2.4.

3.2.7 Non-Physical Actions

Sections 3.2.2 through to 3.2.6 describe physical flood actions. Three forms of non-physical flood actions exist: chemical actions, nuclear actions, and biological actions. Some overlap exists between physical flood actions and non-physical flood actions.

As illustrated in Section 2.4, flood damage to residences is frequently estimated by considering only the chemical actions which occur when water contacts an object. An example of a chemical (contact) action is rusting. Physical consequences may result too, such as timber floor boards warping. Additionally, flood-induced humidity may cause damage, even if flood water does not contact the damaged residence (Crichton, 2002). The chemical (contact) action would be from water vapour rather than water.

Flood water may be contaminated with sewage, petrol, oil, paint, household cleaners, or industrial chemicals. Any corrosiveness or flammability in the contaminants could result in chemical damage to residences. A full propane tank or a vehicle's petrol tank colliding with a residence may result in an explosion. Additionally, the vapour from flood water contaminants may cause damage even if the contaminated flood water does not contact the damaged residence.

Water is a good conductor of electricity. This chemical action in the form of energy is frequently fatal in floods and has the potential to produce electrochemical reactions which damage residences; for example, by breaking down render or paint.

Nuclear actions would be much rarer, occurring, for example, if nuclear fuel becomes a contaminant. One possibility for eastern England is the nuclear power plant sited just above the shingle beach at Sizewell, Suffolk. A highly improbable but catastrophic event during which a storm surge severely damages this structure has a small potential for imparting nuclear actions during the flood.

Biological actions include microorganisms which thrive in damp conditions, particularly moulds and fungi. Floods could also bring animals in contact with residences which normally would not be encountered, such as jellyfish. Macrovertebrates—including fish, alligators, crocodiles, sharks, and snakes—may impart significant physical forces on residences if these animals are brought by a flood into the proximity of residences.

3.2.8 Summary and Interactions

To summarise, the proposed typology for flood actions on residences is:

1. Hydrostatic actions (actions resulting from the water's presence):
    - Lateral pressure from f_{diff}.
    - Capillary rise.
2. Hydrodynamic actions (actions resulting from the water's motion):
    - Velocity: moving water flowing around a residence imparting a hydrodynamic pressure.
    - Velocity's localised effects, such as at corners.
    - Velocity: turbulence.
    - Waves changing hydrostatic pressure.
    - Waves breaking.
3. Erosion Actions (water moving soil; the water's boundary becomes dynamic and moves into the adjacent solids).
4. Buoyancy action: the buoyancy force.
5. Debris actions (actions from solids in the water):
    - Static actions.
    - Dynamic actions.
    - Erosion actions.
6. Non-physical actions.
    - Chemical actions (including contact).
    - Nuclear actions.
    - Biological actions.

Interactions and combinations must also be considered. For example:

- As implied in Section 3.2.5, uplift due to buoyancy reduces the lateral hydrodynamic pressure from v required to move a residence (Black, 1975; Sangrey *et al.*, 1975).
- Chemical actions may corrode materials such as brickwork, glass, timber, or PVC, reducing the ΔP from physical actions required for failure.
- Water saturation may soften materials such as finishes or timber, reducing the ΔP from physical actions required for failure.

- Physical parameters such as flood water pressure and temperature at a point in the flood influence the rates and consequences of chemical reactions, such as explosions.
- Chemical reactions, such as acid mixing with water, influence physical parameters such as flood water temperature and density.
- Contaminants such as sewage or oil produce floods with differing physical and chemical properties.

Larson (1999) describes other noteworthy phenomena. Although the effects fall into the categories set out here, they are important enough to highlight separately. During a storm surge flood, a sudden shift in wind direction may permit the sea to swiftly drain away in a phenomenon known as an ebb surge. The physical forces and pressures created by the sudden retreat of water can exceed those imparted by the ingress of water into a community. As well, powerful gusts of wind, spikes far above the 3 s mean, may destabilise a residence under pressure from the flood's lateral pressures. Finally, wind-driven rain could be powerful enough to cause severe damage.

3.3 Scoping the Problem

3.3.1 Choosing Actions to Study

Lateral pressure from f_{diff} and v shows trends at various spatial scales, including at the community or city level. f, f_{diff}, and v or their ranges may be estimated, such as with the models used by Brown (2003), and a reasonable level of predictability at useful spatial and temporal scales can be achieved. This predictability leads to information helpful for estimating medium-to-large-scale damage and loss patterns in a flood event. These lateral pressures also have a high relevance to the damage outcome from a flood since they could lead to structural failure.

Analysis of these actions should not ignore chemical actions from water contact, i.e. including f through damage curves for slow-rise flood depth. The reason is that if structural collapse does not occur due to the lateral pressures, then damage would occur due to f. Additionally, the lateral pressures may cause only non-structural damage, such as breaking glass, so damage due to f could still be important. When structural failure does occur, though, the impact from f_{diff} and v may dwarf any impact from f alone. The detailed investigation needed to determine

the relative impacts of both the lateral pressures and f would be achievable at a spatial scale the size of a community or city with appropriate models.

Buoyancy's relevance varies. Timber residences may float if not anchored properly (Black, 1975; Sangrey *et al.*, 1975), but masonry residences tend to be securely anchored and would not usually float. Mobiles/caravans are so poorly fastened that they frequently float, becoming debris hazards (e.g. FHRC, 1983). Buoyancy is also relevant for vehicles, people, trees, boats, household items, and other such potential "debris".

Capillary rise's relevance varies. When a storey is flooded, the repair process is generally to completely redecorate the entire storey, irrespective of how high the water rose above f (e.g. Figure 3.3.1.1). In such cases, the effect of capillary rise is negligible. Damage from chemical (contact) action due to f would supersede damage from capillary action.

Figure 3.3.1.1: Apparent Capillary Rise Inside a Flooded Residence, Keighley, U.K., November 2000 (Plaster over the entire storey has been stripped away)

Other cases may occur where capillary rise transcends storeys, although none were found in the literature or in the field. If such cases were to occur, then damage could increase substantially as a result of capillary rise. For example, Crichton (2002) notes special cases of cavity wall insulation damaged by capillary action along with "strawboard wall boards which can have a capillary effect of several metres". As well, damage in cases where flood water does not directly enter a property could be exacerbated by capillary rise. No such cases were documented in the literature, but capillary rise is not a phenomenon specially looked for in flood damage assessments. Another possibility is that damp-proof membranes in good condition prevented capillary rise from causing extensive damage in these cases. To establish the effect of this phenomenon, more attention could be paid to instances of flood damage claims where flood water has not entered a residence.

Even in cases in which capillary rise has the potential for augmenting flood damage, swift action in drying a property could prevent much of the damage. Similarly, any structural collapse would obscure capillary rise effects and would supersede capillary rise damage.

Damage from the remaining actions—including erosion, debris, turbulence, non-contact chemical actions, nuclear actions, and biological actions—may be absent or may be the most prominent cause of damage for a given flood scenario. Their specific impacts are generally highly localised in space and time. For example:

- A gate being opened or closed, and thereby impeding or permitting high v down a slope, could make the difference between scour undermining a foundation or not.
- A car impacting one house and exploding could damage it leaving its neighbours unscathed even though all properties experience similar f, f_{diff}, and v.
- Swift drying and thorough disinfecting of a flooded residence prevents mould (EA/CIRIA, 2001a).

More generally, non-structural damage from these actions would be superseded by structural damage from lateral pressures. Structural damage from these actions, however, could supersede structural damage from lateral pressures. The transition between structural and non-structural damage for these actions occurs at a highly localised level. Therefore, larger-scale estimation of damage from such actions is difficult, although for each action, scenarios could be assumed and the consequences could be investigated.

From this discussion, the actions are classified in the following categories with respect to relative importance for residence flood damage assessment:

- Lateral pressure from f_{diff} and v along with water contact from f should be considered in estimating damage from most floods. These actions frequently have high relevance and can be predicted in a manner applicable to large-scale loss analyses.
- Buoyancy can be predicted for large-scale loss analyses but its relevance varies depending on the residence or residence component affected by the flood.
- The relevance of the remaining flood actions (capillary rise, erosion, debris, turbulence, waves, other velocity actions, other chemical actions, nuclear actions, biological actions) varies depending on the exact form in which the action appears and the consequences from other potentially relevant actions. These actions frequently cannot be predicted in a manner applicable to large-scale loss analyses.

The duration of each flood action must also be considered in analysing its impact.

Given these issues, lateral pressures from f_{diff} and v plus water contact due to f offered the best potential for a study contributing to as wide a range of applications and first-order results as possible. f, f_{diff}, and v are standard outputs from hydrodynamic models, such as those used by Brown (2003), adding impetus to a detailed investigation of these parameters' impacts on flood damage to residence. Therefore, these actions were selected for the main focus of this study. Since f_{diff} and v have not been examined in detail before, the emphasis will be on them. This selection ensures that a new contribution to knowledge shall be made by this study.

Avoiding detailed analysis of many of the flood actions described introduces uncertainties into any damage estimation or analysis. For example, as noted in Section 3.2.3, wave pressures may exceed other hydrostatic and hydrodynamic pressures by at least an order of magnitude, so waves may not be ignored if they are expected to occur. Additionally, other actions may not be entirely neglected due to their potential importance in certain scenarios. They are mentioned, along with their specific impacts for the scenario being examined, when appropriate. Buoyancy, for example, could have potentially been as relevant as other actions, but as illustrated in Chapter 6, observed residences were masonry and thus are rarely susceptible to buoyancy.

As residence flood damage is examined in more detail, more data and experience become available. Observed damage could be matched with estimated damage to better quantify both the damage and the uncertainties and to identify the origin of the uncertainties. The principal factors leading to uncertainties may then be extracted and targeted for detailed analysis. Such knowledge could be applied to developing modifiers to damage functions, such as a factor by which to multiply damage depending on the set of actions which are present in the specific circumstances. The current tendency to focus on damage from f due to slow-rise floods provides a needed basis, but a more comprehensive approach would be useful for flood damage estimation and analysis. This study's focus on f, f_{diff}, and v starts this process and provides a needed contribution, but is not the end of such studies.

3.3.2 Applying the Analysis of Actions

With the focus established on the lateral pressures from f_{diff} and v plus water contact due to f, a preview of the form which the results would take is appropriate. A practical, repeatable, useful, and usable output is sought. This vulnerability study should further seek to use the

results from Brown's (2003) hazard study. Aiming for this integration, while not an overriding objective for producing an adequate vulnerability analysis, is key to adding to the understanding of how risk may be quantified using the limited tools available.

The desired output from this study must meet the objectives and subobjectives outlined in Chapter 1. A method for profiling the vulnerability of a residence or a group of residences with respect to f_{diff} and v must be established to link this study's objectives with the discussion in this Chapter. The most straightforward approach is a matrix, with f_{diff} on one axis, v on another axis, and a statement of vulnerability at each f_{diff}-v combination (Table 3.3.2.1). The consequences of f may be integrated into this matrix in cases where structural failure does not occur.

Table 3.3.2.1: Template for First-Order Vulnerability Profile

v ↓	f_{diff}				
	0.0 m	$f_{diff\,1}$	$f_{diff\,2}$	$f_{diff\,3}$	[…]
0.0 m/s					
v_1					
v_2					
v_3					
[…]					

The capability of the hydrodynamic models used by Brown (2003) which determine f, f_{diff}, and v extends to providing highly precise values. For practical purposes, Brown (2003) suggests that the smallest intervals which would produce credible results would be approximately 0.5 m for f and f_{diff} and approximately 0.5 m/s for v. Although the accuracy could be reported at a higher resolution, the lack of error bars associated with the models' output would make interpretation difficult (Brown, 2003). The relatively wide spacing of the 0.5 m and 0.5 m/s intervals provides clear pigeonholes for vulnerability statements, thereby assisting in reducing the degree of uncertainty associated with each f_{diff}-v combination. As the confidence in the models' accuracy improves, further rows and columns could be added to the vulnerability matrices to give results at a higher resolution.

Assumptions which must be made to develop these vulnerability matrices are:

- The flood water reaches f = f_{diff} and v instantaneously at a point in space.
- No water had contacted the residence before that f_{diff}-v combination is reached.
- Damage arises due to water, which may be saline, but due to neither debris nor contaminants.

- v is perpendicular to the residence component under consideration. If the angle of incidence is less than $\pi/2$, i.e. away from the perpendicular in any spherical direction, then to obtain an impact velocity v, the true flood velocity would need to be v divided by the sine of the angle of incidence. If this angle is 0, then the flow is parallel to the residence and the flood velocity does not directly impact the residence.
- The lateral pressure from v is the same between $y = 0$ and $y = f_{diff}$.

As well, extracting the maxima for f_{diff} and v for a flood scenario would be important for determining the maximum feasible damage. This approach entails the assumptions that for a point in space, $f_{diff\ max}$ and v_{max} coincide in time and that these values are reached instantaneously at the start of the flood. These assumptions are relatively realistic since a fast-rise flood would be expected to be highly turbulent and to be moving rapidly at the beginning, thereby immediately generating a large f_{diff} coinciding with a large v. The relatively large 0.5 m and 0.5 m/s intervals for f_{diff} and v respectively also help in making the maxima coincide. This worst-case scenario of the maxima coinciding would predict the maximum feasible damage.

Damage from water contact, dependent on f, enters the vulnerability matrices when the f_{diff}-v combination would not cause structural damage. A scenario with $f_{diff} = 0$ but $f,v > 0$, however, could also yield structural damage. This situation is not explicit in the vulnerability matrices (Chapter 12), but is discussed in the application and analysis of the vulnerability matrices (Chapter 13).

The statement of vulnerability for each f_{diff}-v combination, determined in Part II, provides an approximate indication of the damage to be expected. The damage could be less if $f_{diff\ max}$ and v_{max} do not coincide in time or if $f_{diff\ max}$ and v_{max} are not reached instantaneously so that $f > f_{diff}$. The damage could be more if other actions such as waves, erosion, debris, or contaminants are present. The vulnerability matrices, at the first order, provide a starting point for, and a new contribution to, determining the failure modes and damage which a residence may experience in a given flood scenario.

This approach significantly bounds the problem identified in Section 2.5. The risk environment examined in this dissertation for residences in coastal, eastern England is plugging the knowledge gap of non-f damage potential at the first order. Nonetheless, a practical output is provided which can be used as a strong starting point for further study.

3.4 Conclusions

The flood actions on residences which are most relevant and most applicable to analysis for loss prediction are the combination of lateral hydrostatic pressure and lateral hydrodynamic pressure imparted by instantaneous f_{diff} and instantaneous v respectively along with damage from water contact due to f. Focussing on these three actions produces a first-order analysis of the physical vulnerability of residences to floods. The uncertainties in this analysis, introduced by not directly considering other actions including waves and corrosion, may be reduced once more data and experience are available by incorporating the other actions into the analysis. Meanwhile, a significant contribution to knowledge and a clear advancement of flood damage prediction and analysis are attained by considering in detail the flood actions of f_{diff}, and v plus f where needed. A method for using the three principal actions to predict residence failure has been proposed involving the development of vulnerability matrices.

The knowledge established in Part I has set the stage for this study. The problem has been defined, justified, placed in context, and scoped in order to identify a key area where an original contribution could be made. Part II outlines the core research of this study for which new methods were developed and carried out to produce the vulnerability matrices described in Section 3.3.2.

Part II

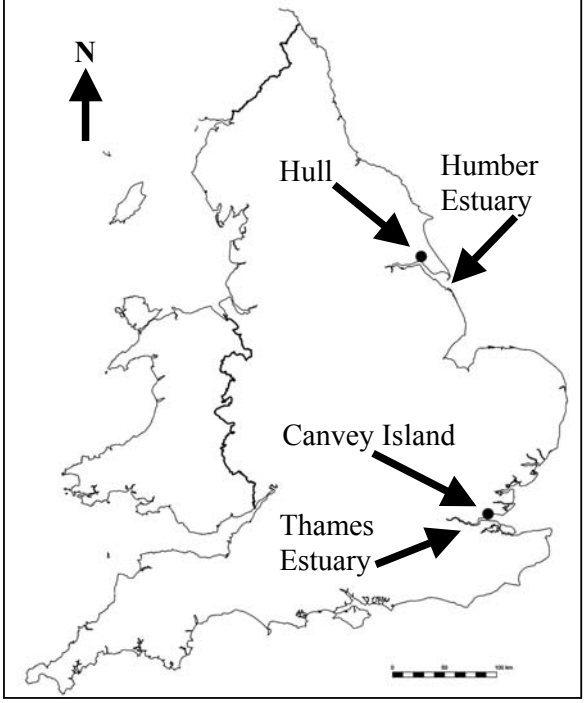
4. Case Study Sites

4.1 Selection Process

Chapter 1 identified coastal, eastern England from the Thames Estuary to the Humber Estuary as this study's area of interest. Two case study sites could realistically be examined within the timeframe available. The criteria considered were:

- Natural environment:
  - -Coastal situation.
  - -Hydrodynamic characteristics
  - -Topographic characteristics.
- Built environment:
  - -Nature and state of sea and flood defences.
  - -Flood vulnerability characteristics of residences.
- Other:
  - -Flood disaster history.
  - -Flood disaster potential.
  - -Interest of different stakeholders, including the insurance industry, in flooding at the site.
  - -Potential for flood actions of interest to manifest, particularly from storm surge.

Figure 4.1.1: England and Wales and the Case Study Sites
(Reproduced from the 1999 Ordnance Survey map with the permission of the Controller of Her Majesty's Stationery Office, © Crown Copyright NC/01/26576.)



Research on the major settlements in the area was completed including several field visits (Table 4.1.1). Smaller communities, including Cleethorpes, Mablethorpe, Dunwich, and Leigh-on-Sea were also visited, but none were of major interest for this study. London and Kent were deemed to be outside the main area of interest. Based on the criteria, the case study sites Kingston-

upon-Hull and Canvey Island were selected (Figure 4.1.1). Both sites are estuarine examples, but enough interest and diversity exists as noted in Sections 4.2, 4.3, and 4.4.

All sites in Table 4.1.1 except Clacton-on-Sea would likely have been worthwhile case study sites. Canvey Island and Kingston-Upon-Hull have all the features required to a much more prominent degree than the other sites without any of the weaknesses present in other places.

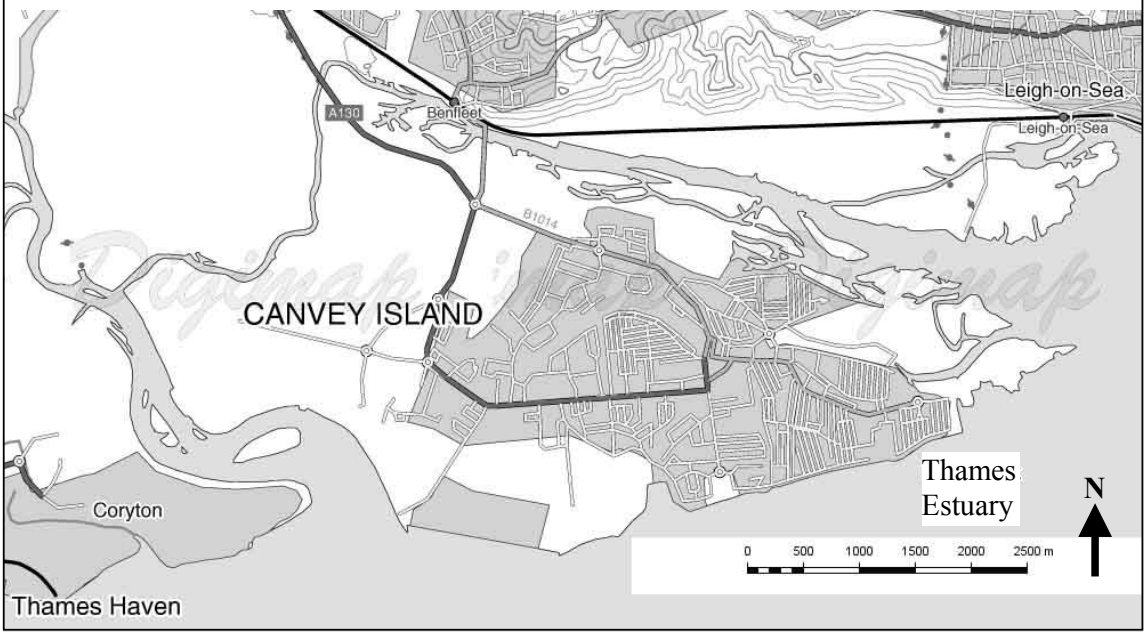
Table 4.1.1: Selecting Case Study Sites

Settlement	Action	Notes
Canvey Island, Essex	Selected	See Section 4.2.
Clacton-on-Sea, Essex	Researched	Vulnerability features not as prominent as at other sites.
Felixstowe, Suffolk	Visited	Of high interest, but large areas are far enough above sea level to have low storm surge vulnerability.
Great Yarmouth, Norfolk	Visited	Of high interest and nearly selected, but data from the Environment Agency which were essential for Brown's (2003) work were not available.
Grimsby, Humberside	Visited	Of interest, but coastal residential areas are less prominent than at other sites.
Harwich, Essex	Researched	Of interest, but less variety of potential flood events than at other sites.
Ipswich, Suffolk	Researched	Of interest, but its location is more protected than other sites.
King's Lynn, Norfolk	Visited	Of high interest and nearly selected, but other sites had features which were of slightly more interest.
Kingston-Upon-Hull, Humberside	Selected	See Section 4.3.
Lowestoft, Suffolk	Visited	Of interest, but most vulnerable areas are not residential.
Maldon, Essex	Visited	Of interest, but not as big as other possible sites. As well, large areas are far enough above the estuary to have low storm surge vulnerability.
Skegness, Lincolnshire	Visited	Of interest, but not as big as other possible sites.
Southend-on-Sea, Essex	Visited	Of interest, but other sites better match the criteria.

4.2 Canvey Island

Canvey Island (Figure 4.2.1) is a flat, low-lying island on the north shore of the Thames Estuary, downstream (east) from London. Remains of Celtic and Roman settlements have been found. In the medieval age, Canvey Island was marshland subject to frequent flooding and used for grazing, but it was drained under the direction of Dutchman Cornelius

Figure 4.2.1: Canvey Island
(Reproduced from the 2001 Ordnance Survey map with the permission of the Controller of Her Majesty's Stationery Office, © Crown Copyright NC/01/26576.)



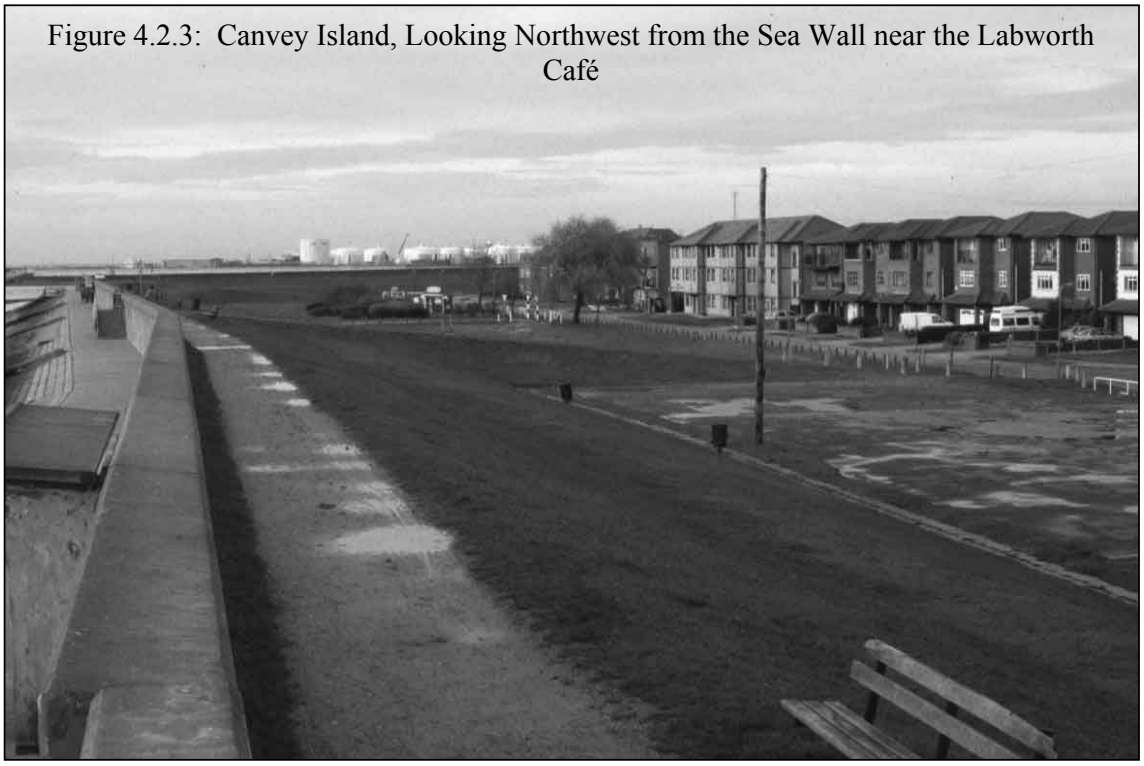
Vermuyden in the seventeenth century. The first sea defences were constructed in 1623 and Dutch settlers formed the first Canvey Island communities of the modern era. The population did not expand rapidly until the 1920s, with 1,795 inhabitants in 1921 but "over 6,000" in 1927 during which time the number of buildings rose from 300 to "about 1,950" (McCave, c. 1928). The 1991 U.K. census recorded 35,700 residents.

Figure 4.2.2: Plaque on Canvey Island

The mean high water mark of the Thames Estuary at Canvey Island is higher than most of Canvey Island's land. The storm surge of 1953 (see Section 1.1) breached Canvey Island's sea defences killing 58 people there (Figure 4.2.2; Grieve, 1959; Pollard, 1978; Summers, 1978).

A concrete wall rising 6.0-7.5 m above Ordnance Datum Newlyn (approximately 3-4 m above the mean high water mark) currently surrounds Canvey Island (Anglian Water Authority, 1981-2; Figure 4.2.3). EA (2001b) claims that until 2030 the minimum level of protection provided by this defence will be against a storm surge level with a probability of 0.001 of being exceeded each year. The defences, however,

Figure 4.2.3: Canvey Island, Looking Northwest from the Sea Wall near the Labworth Café



display signs of deterioration including significant cracks in the concrete and disintegrating seals between wall slabs. Metal access gates along the south side of Canvey Island represent a further weakness since closure would be challenging during extreme weather conditions. As well, closing the gates requires inserting metal bolts into holes in the wall but debris clogs many of these holes while some rubber seals at the gates' undersides imperfectly seal the gap with the wall.

Canvey Island relied on ferries to reach the mainland until the opening of the Colvin Bridge on 21 May 1931. Two roads currently connect Canvey Island with the mainland, but both travel through the same roundabout just before departing (see Figure 4.2.1). Thus, evacuation by land vehicle during an emergency is limited by this roundabout's capacity. No railway exists on Canvey Island.

Although Canvey Island is predominantly residential, the western part remains undeveloped while the southwest is dominated by heavy industry and commercial facilities. Wharves for oil companies lie at the eastern fringes of this industrial estate (see the background in Figure 4.2.3). Just off-island to the west, a large oil refinery and oil storage depot sits on the Thames shoreline. Should a flood occur, further devastation may occur from the on-island and off-island anthropogenic hazards (HSE, 1978).

Planning is fragmented with large gaps between some houses and almost no space between others. The rate of replacement for residences is high but without an obvious, systematic approach. The north part, almost uninhabited in 1953, is now covered with properties of various sizes. In 2002, the caravan park in the northeast was turned into a new development of residential homes. The caravan park in the south was also being removed to make way for a new development.

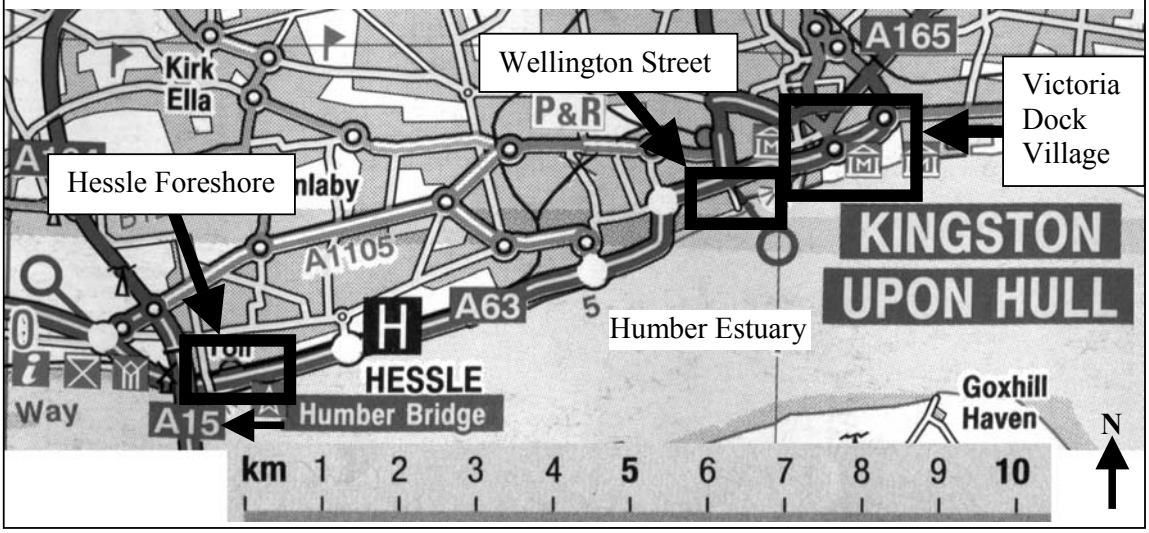
Further information on Canvey Island appears in Table 4.4.1 and the residence survey (Chapter 6) provides details of Canvey Island's residences. On Canvey Island, all residences were deemed to be vulnerable to the flood actions of ΔP from f_{diff} and v, thus areas over the entire island were selected for the survey.

4.3 Kingston-Upon-Hull (Hull)

Hull (Figure 4.3.1) is situated on the north shore of the Humber Estuary on an alluvial plain. The city is built on drained salt marsh with many residential areas lying more than 1.8 m below the highest recorded tide (East Riding of Yorkshire Council, c. 1999; HCIS, 1991; Stapleton, 1994). Hull's status as a major English port was achieved during the medieval age, when it was used primarily for shipping wool. Edward I granted the city its charter as King's Town Upon Hull in 1299 to recognise its strategic importance.

The city's recent history includes at least thirty recorded flood events since 1850. Storm

Figure 4.3.1: Areas of Hull with Residences Most Vulnerable to Flood Actions
(Reproduced from the 2001 Ordnance Survey map with the permission of the Controller of Her Majesty's Stationery Office, © Crown Copyright NC/01/26576.)



surges damaging residential areas occurred in 1921, 1961, and 1969 amongst other years. Interest in residence flood vulnerability in Hull appears to have slowly diminished with the increasing desire for new development: "Around Hull, older properties are built on higher ground and modern properties on lower. You can actually underwrite Hull flood risks based on the age of the properties" (Crichton, 2002).

The Humber Estuary catchment drains one fifth of England's area and the estuary's maximum tidal range is large (7.1 m), coupling with a strong current and major silting to create a complex hydrodynamic environment (Stapleton, 1994). Some uncertainty exists regarding the estuary's potential flood actions. For example, Townend (2001) stated that waves during a storm surge would not be a concern for Hull but Norton (2001) noted that the issue has not been well studied. Pelleymounter (1994) writes "Although the impact of wave action reduces up the estuary, effects remain a significant factor up to Trent Falls", upstream from Hull.

Anthropogenic hazards near residential areas include BP's chemical works on the eastern side of the city and petrochemical tanks around the docks. A further natural hazard exists from the River Hull, which flows through the city centre before reaching the Humber Estuary, yielding the possibility of conjunctive flood events. The Hull Barrage (Figure 4.3.2) opened in 1980 just upstream of the confluence with the estuary and adds 1.05 m to the storm surge level which could occur without flooding the city centre due to propagation up the River Hull (HCIS, 1991). When the Hull City Council's planning department was visited on 7 December 1999, they could not explain who owns and operates the Hull Barrage. Further investigation confirmed that the Environment Agency (England and Wales) is responsible.

Hull's population from the 1991 U.K. census is 242,200. This size is reflected in the economic diversity and types of residences observed throughout the city. The shipping industry's decline in recent decades has led to the closure of some of Hull's docks. The main replacements are commercial and residential properties, mostly built since 1990.

The Humber Estuary has a tidal defence strategy (Pelleymounter, 1994), but within Hull the sea defences are diverse. A shingle bank raised approximately 2 m above the mean high water mark protects a residential community just east of the Humber Bridge (Hessle Foreshore; Figure 4.3.3). A vertical concrete wall raised approximately 4 m above the mean high water mark protects a retail and leisure park just west of the city centre (St. Andrew's Quay). The residential community of Victoria Dock Village, at the point where the Humber

Estuary narrows, just east of the River Hull, is protected by a concrete sloping wall approximately 2 m above the mean high water mark (Figures 4.3.2 and 4.3.4). This wall stops approximately 200 m from the east end of the community, leaving a shingle bank including a decaying timber boat as the rest of the sea defences.

Figure 4.3.2: Hull Barrage
Viewed from Victoria Dock Village

Figure 4.3.3: Shingle Bank at Hessle Foreshore
Looking East

Figure 4.3.4: Defences Along Humber Estuary at
Victoria Dock Village

Further information on Hull appears in Table 4.4.1 and the residence survey (Section 6) provides details of Hull's residences which are most vulnerable to the flood actions of ΔP from f_{diff} and v. Three main, residential communities along the Humber Estuary exist and these were selected for the residence survey. From west to east, they are Hessle Foreshore, Wellington Street, and Victoria Dock Village (Figure 4.3.1).

4.4 Summary and Conclusions

The characteristics of the case study sites which make them appropriate for this study are summarised in Table 4.4.1. Canvey Island and Hull fulfil the criteria set out in Section 4.1 by providing coastal, urban areas which are vulnerable to flooding, particularly the flood actions of ΔP from f_{diff} and v. Different sea and flood defences are present along with complex hydrodynamic conditions. The residences must be characterised in detail to determine their vulnerability to the flood actions of interest. The method of the residence survey is discussed in Chapter 5 and the results are presented in Chapter 6.

Table 4.4.1: Summary of Characteristics of Canvey Island and Hull

Criterion	Canvey Island	Hull
Coastal situation.	An island sited in the mouth of an estuary.	Sited where a river meets an estuary.
Hydrodynamic characteristics.	Estuary has a large catchment, a large tidal range, strong currents, and a large sediment load.	Estuary has a large catchment, a large tidal range, strong currents, and a large sediment load.
Topographic characteristics.	Flat, lying below the mean high water mark. Sand flats lie between Canvey Island and the open sea.	Flat, with much area lying below the highest recorded tide. A storm surge must propagate up the Estuary around some curves.
Sea and flood defences.	Surrounded by substantial structural defences but the defences have weaknesses.	A variety of defences, some concrete, some shingle. A barrage sits at the mouth of the River Hull to prevent a storm surge from the Humber Estuary propagating upstream.
Flood disaster history.	Prior to modern settlement, frequent sea flooding. Devastated by the 1953 storm surge.	Frequent flooding and significant storm surge events during the past century.
Flood disaster potential.	High from storm surge. Heavy rainfall could also overwhelm the drainage system or overflow from areas of standing water.	High from storm surge, river flooding, or conjunctive events.
Residences in relation to principal hazard parameters.	Residences lie below the mean high water mark. The sea defences are walls, hence breaching, overtopping, and undermining are possibilities. f_{diff}, v, and waves could impact on residences. Water could come from any direction.	Residences lie above the mean high water mark. The sea defences form the shoreline, so breaching is not possible but overtopping and undermining could occur. v and waves would be the main impacts on residences. Significant f_{diff} could occur, but would be unlikely. A conjunctive event with river flooding implies that water could come from any direction.
Residence physical vulnerability.	Vulnerable to flood actions across Canvey Island. Strong potential for secondary hazards following a flood event.	Three new communities built adjacent to the estuary which have not yet experienced a major flood event. Strong potential for secondary hazards following a flood event.

5. Residence Survey Method

5.1 Introduction

The purpose of this survey is to identify the features of residences which are most physically vulnerable to the flood actions of ΔP from f_{diff} and v. The method is tailored to this specification. To establish this first-order level of vulnerability, potential routes of flood water infiltration into the structure and potential mechanisms of failure must be identified.

The method must also be doable and repeatable without excessive resource expenditure. This efficiency feature assists in formulating a method relevant to homeowners, local authorities, and the insurance industry so that they can determine flood vulnerability of a residence or a community. Despite these specifications, the concepts and methodological principles could be applied to non-residential structures and for determining vulnerabilities to perils other than flooding.

To meet these criteria, the residence survey examined features identified externally and from public land. This external viewing defines the difference between "a residence-by-residence survey" and "a residence-to-residence survey" which may imply interviewing residents in person, by telephone, or by post. Only certain data are available from a residence-by-residence survey, but the importance of this approach is to provide a workable, legal method for a variety of end users. Although privacy violations may possibly be claimed due to the method presented, no action was illegal and any privacy concerns apply equally to any individual on public land.

5.2 Previous Studies

Few studies exist which could have been emulated for this survey. The most useful study was done by Cambridge Architectural Research Limited for Risk Management Solutions, Inc. This unpublished study comprised several surveys undertaken from 1996 to 1998 of various types of buildings, streets, and communities in England. The focus was on identifying vulnerabilities which would interest insurers. The main vulnerability was to theft, but other perils including natural hazards were considered in the survey design and could be considered from the survey results.

Risk Management Solutions, Inc. has also developed its own U.K. Residential Building Stock Profile. This profile is a corporate product designed to integrate with their U.K. Peril Rating Profiles, which includes flood, to predict "vulnerabilities and loss ratios" for the insurance industry (RMS, 2002). The "residential building stock profile is a model of the number, age, and insurable value of the typical residential building types that make up the over 24 million dwellings in the U.K." (Berger, 2000). Six building types—detached, semi-detached, terraced, bungalow, converted flat, and purpose-built flat—are provided, derived directly from the 1991 U.K. Census, although "bungalows are not distinguished in the 1991 Census, and therefore the proportion of bungalows was inferred based on data from the English Housing Condition Survey [see Table 5.2.1]" (Berger, 2000). Three age categories—pre-1918, 1919-1944, and post-1945—are provided, "derived by modelling typical town age profiles using town age maps, data from the 1991 U.K. Census and the English Housing Condition Survey" (Berger, 2000). A residence's insurable value is calculated from various sources as the estimated rebuilding cost.

For another building study, Penz (1983) estimated the potential for, and savings from, retrofitting existing residential buildings in order to save energy through the use of passive solar heating. He provides a detailed description of his housing survey of 413 dwellings undertaken in Cambridge, U.K. which used six age categories (pre-1900, 1900-1918, 1919-1939, 1940-1963, 1964-1970, and post-1970) and eight type categories (terrace, terrace end, semi-detached, detached, bungalow, flat > 5 floors, flat < 4 floors, and maisonette or walk-up flat). Penz (1983) also provides data about floor areas, wall areas, and room layout, which he obtained by randomly selecting a subsample of the externally surveyed houses and conducting internal surveys and occupier questionnaires.

For natural hazards, Spence *et al.* (2000) supervised a team which conducted a building survey in Naples, Italy to determine the population's vulnerability to pyroclastic flows from a possible eruption of Mount Vesuvius. The typology of casualties provides helpful patterns for characteristics of buildings which could permit natural hazards such as pyroclastic flow gases or flood water to enter.

Approximately every five years, House Condition Surveys are undertaken in the U.K. with separate surveys for England, Northern Ireland, Scotland, and Wales. The surveys collect comprehensive data on the physical condition of housing and the socio-economic characteristics of occupants for a subset of residences within their jurisdiction. The English

House Condition Survey uses a questionnaire approximately 25 pages in length, with the exact questions asked depending on the type of building being surveyed. The most recent surveys are listed in Table 5.2.1. These surveys indicate the data which could be collected by a survey. Discussions (e.g. DOE, 1993; NIHE, c. 1998) include descriptions of data quality and methodological problems encountered.

Table 5.2.1: Most Recent U.K. House Condition Surveys

Housing Condition Survey	Year Conducted	Year Published	Website
England	2001	2003	http://www.housing.dtlr.gov.uk/research/ehcs/index.htm
Northern Ireland	1996	1999	see http://www.nihe.gov.uk/Research/reports.asp
Scotland	2002	2003	http://www.shcs.gov.uk
Wales	1998	2001	http://www.wales.gov.uk/keypubstatisticsforwales/content/publication/environment/2001/whcs/intro.htm

Smith and Greenaway (1994) provide an example of an Australian building survey used for assessing potential storm surge damage to both residential and commercial properties:

- Property type was recorded as detached, terrace [sic], or flats/apartments.
- Size was recorded as small including all flats, medium representing a three-bedroom house, and large which generally had four or more bedrooms.
- Number of storeys was recorded as 1 or 2+.

Smith and Greenaway (1994) note that roof data were collected for consideration of cyclone damage. Other studies examined (e.g. Schill and Scafidi, 1997) illustrate that socio-economic data may relate to flood vulnerability, such as the number of visible maintenance deficiencies and the percent of houses with boarded up buildings nearby.

5.3 Residence Definition

The survey method for this study, including the classes by which residences are defined, must take into account the specific vulnerabilities to ΔP from f_{diff} and v and must consider the characteristics which could be observed externally. Past studies (Section 5.2), including those specifically on flood damage (Section 2.4), indicate neither how each residence class defined by age and configuration has different vulnerabilities to the flood actions considered nor why the vulnerabilities should necessarily be different.

Regarding residence age, an older well-maintained residence could be less vulnerable to specific flood actions than a newer poorly-maintained or poorly-built residence. Maintenance is one disaster mitigation tool, thus state of maintenance would be more relevant than age for suggesting potential damage which could exacerbate flood damage. Age may provide clues regarding how the house was constructed and the materials used, but direct observation, if possible, would be preferable to using this proxy. Therefore, age was not used as a defining variable for residence class, but age data were collected.

A further concern arose regarding the definition of residence configuration which in past studies (see Section 5.2 and the U.K. studies from FHRC described in Section 2.4) involved arbitrary categorisations. For example, a terraced middle residence has obvious differences in configuration from a terraced end residence, yet some studies group them together. Terraced ends are much more similar to semi-detached residences. As well, flats are often considered separately as one category, yet an end flat which has three exterior walls has obvious differences in configuration to a middle flat with one or two exterior walls. Another difference exists between residences with two adjacent walls and with two opposite walls, yet these categories are neither discussed nor separated in previous studies. Finally, identifying bungalows as an entirely separate category, as done in some studies, is arbitrary considering that one-storey terraced or semi-detached houses are not separated out. Instead, the number of storeys or levels would seem to be an important defining characteristic for any residence, a fact confirmed by analysing wall failure (Chapter 9).

In developing a survey method for determining first-order, physical vulnerability to ΔP from f_{diff} and v, the standard age-configuration approach is inadequate. Instead, based on the discussion above, residence classes were defined by (a) configuration of exterior walls (after Steadman *et al.*, 1991) and (b) number of storeys for houses or level for flats (Table 5.3.1). For flats, the number of storeys must be noted too. Further definitional notes appear in Section 5.5 and the revision of this method following field work is discussed in Chapter 6.

Table 5.3.1: Template for Defining Residence Classes

Configuration for Houses	Max. # of storeys →	1	2	3	4
	houses, 1 exterior wall				
	houses, 2 adjacent exterior walls				
	houses, 2 opposite exterior walls				
	houses, 3 exterior walls				
	houses, 4 exterior walls				
	houses, other configuration				
Configuration for Flats	Level →	ground	1st floor	2nd floor	3rd floor
	flats, 1 exterior wall				
	flats, 2 adjacent exterior walls				
	flats, 2 opposite exterior walls				
	flats, 3 exterior walls				
	flats, 4 exterior walls				
	flats, other configuration				
caravans/mobiles (all are expected to be 1 storey with 4 exterior walls but any variation would be recorded)					

The following labels for exterior wall configuration are used:

- w1: 1 exterior wall
- w2adj: 2 adjacent exterior walls
- w2opp: 2 opposite exterior walls
- w3: 3 exterior walls
- w4: 4 exterior walls

5.4 Survey Method: Data Collected

Not all residence characteristics impact the physical vulnerability to floods. The data collected achieved a balance between (a) obtaining all data needed without repeating surveys in each location and (b) ensuring that the data are collectable, analysable, and relevant. An analogous trade-off was between the number of residences surveyed and the amount of data per residence.

Two levels of surveying were completed. The first level was a quick survey (Section 5.4.1) for over 1,000 residences in each case study site to record the main defining qualities of residence vulnerability to ΔP from f_{diff} and v, i.e. the configuration as defined in Table 5.3.1. This rapid house-by-house census was completed in representative areas throughout each case study site to be indicative of the residential building stock.

The second level was a photographic survey (Section 5.4.2) of several hundred residences representative of the entire residential building stock. This survey involved taking photographs of all visible walls of a residence and using the photographs to extract relevant vulnerability characteristics. Again, a representative sample of each case study site was covered by including in the photographic survey the same proportion of residences from each class as were observed during the quick survey.

The original idea was that vulnerability characteristics gleaned from the photographs for several residences of a specific class would represent that class. The distribution of values from the sample for a vulnerability characteristic (e.g. a histogram of storey heights) would be taken to apply to the entire population of residences in each class. Knowing the vulnerability characteristics representative of each class along with the proportion of each residence class present leads to a comprehensive understanding of the vulnerability characteristics in each case study site. The revision of this method during data analysis is discussed in Section 6.

5.4.1 Quick Survey

The main fields examined for the quick survey (Table 5.4.1.1) reflect the principal criteria which express a fundamental difference amongst residence classes with respect to first-order vulnerability to ΔP from f_{diff} and v.

Table 5.4.1.1: Quick Survey Data

Subclass	Fields	Importance
Residence Class	See Table 5.3.1	See Section 5.3.
Relative Height	To the nearest 0.1 m.	To determine the depth of flood water encountered by the residence.
Year Built	As accurate as possible from the OS Historical Maps (various dates).	To determine age and construction style, such as foundation type.
Design Purpose	commercial, garage, public, residential, stables, warehouse (goods storage)	To determine what structural changes the residence may have undergone.

5.4.2 Photographic Survey

The photographic survey was completed for the items in the following tables. The "Fields" column is representative, not comprehensive. Unlisted entries under "Fields" were recorded if observed. Similarly, rather than using the ambiguous terms "other" or "combination", the specifics were recorded for each case. For the "Importance" column, details on the following items are found in the affiliated Chapters:

- Flood Rise Rate: Chapter 8
- Wall Failure: Chapter 9
- Glass Failure: Chapter 10
- Other Failure Modes: Chapters 7 and 11

Data on roofs were collected but were not used because they do not contribute to first-order vulnerability to the flood actions considered (Chapters 7 and 11). These roof data are not described here nor in the survey results (Chapter 6).

Table 5.4.2.1: Residence Properties

Subclass	Fields	Importance
Garage	# cars	For properly calculating A and Π.
Basement	yes/no	Identifies potential weaknesses.
Loft used for living	yes/no	For properly recording j.
Storeys	for houses: # for flats: # out of #	Affects flood characteristics experienced and for wall failure.
Wall Material (separate entry for each wall)	brickwork, blockwork	Wall failure.
Wall Finish (separate entry for each wall)	none obvious, painted, pebble dash, render, stone facing, tile, timber or plastic facing	Identifies potential weaknesses.
Wall Condition (separate entry for each wall)	no damage visible some damage unrepaired significant damage unrepaired	Identifies potential weaknesses.
Wall Worst Damage Visible	brick damage cracks erosion finish damage mortar damage non-verticality water marks or stains	Identifies potential weaknesses.
Cavity Walls	yes/no	Wall failure.

Geometrical data (Figure 5.4.2.1; Table 5.4.2.2) were measured by counting bricks and, rarely, blocks, with the brick sizes corroborated by field measurements. Some photographs included a field assistant whose measurements were known as an additional method of checking the values. The number of horizontal bricks counted was multiplied by 0.225 m and the number of vertical bricks counted was multiplied by 0.075 m. This close agreement with the working sizes of bricks (BS 3921:1985) arose from the modern age of the residences (see Sections 6.1.10 and 6.2.10). Blocks were measured in the field and were rare enough that measurements were completed for each case.

Table 5.4.2.2: Geometrical Data

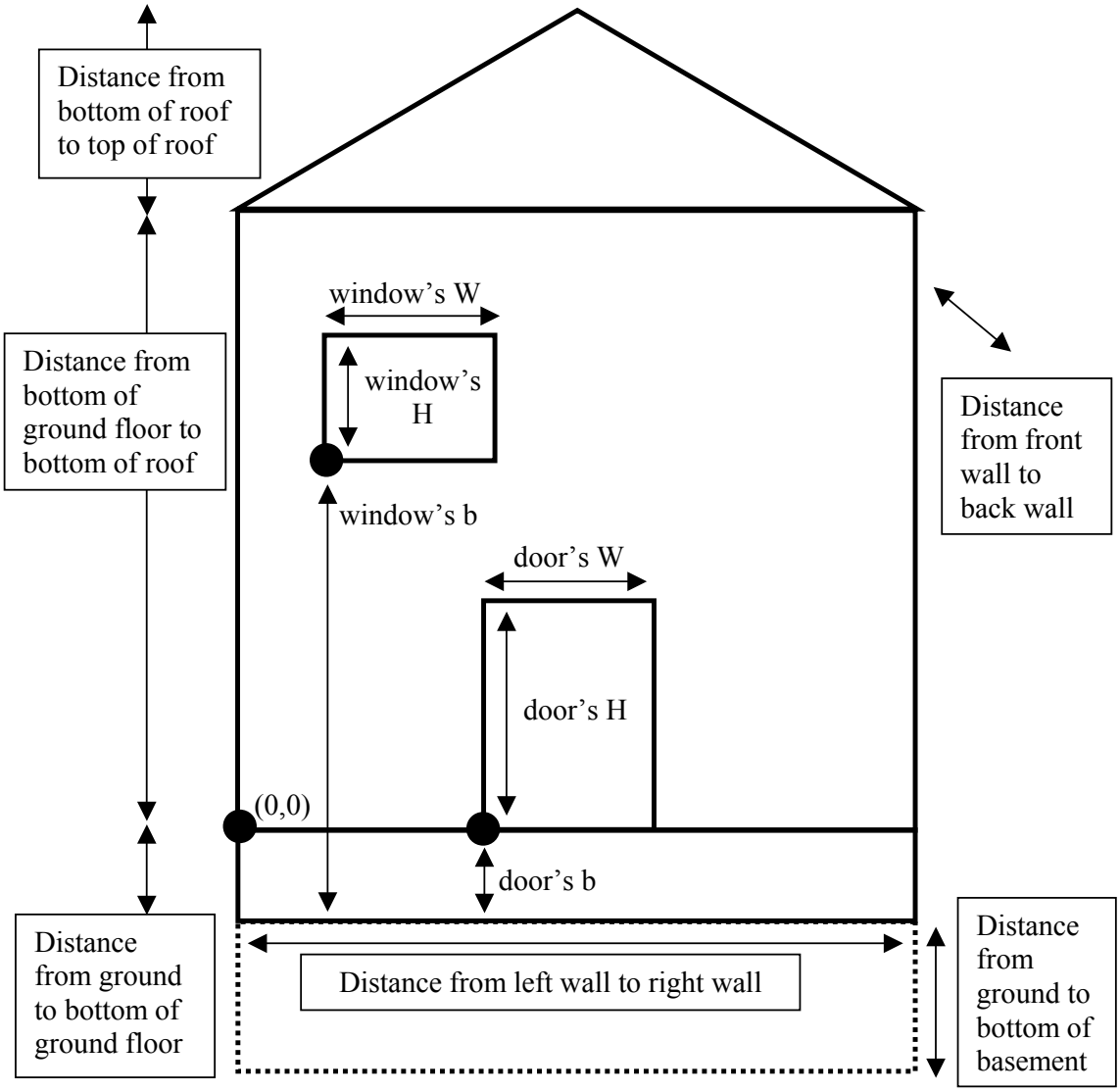
Measured Field	Accuracy	Source
ground to ground floor	± 1 brick ≈ ± 0.1 m	Photographic survey.
ground to basement bottom	± 1 brick ≈ ± 0.1 m	Photographic survey.
ground floor to roof bottom	± 1 brick ≈ ± 0.1 m	Photographic survey.
plan area minus garage	< ± 1 m per wall length	Digimap (2001-2) and OS (2000) minus garage area of 15 m^2 (Adler, 1999).
exterior perimeter	< ± 1 m per wall length	Digimap (2001-2) and OS (2000).

Cooke (1996) describes the standard method for calculating errors when combining variables. The error in Π is less than ± 2 m. The error in A is less than $2^{1/2}$ multiplied by the average wall length of the residence, yielding approximately ± 7 m^2 for small residences and approximately ± 14 m^2 for large residences. These calculations assume that a residence's wall lengths are not correlated, even though the expectation is that some correlation exists. Since the standard error combination formulae, even for correlated variables, are approximations (Cooke, 1996) and only an estimate of the error is needed, this approximation is acceptable.

Each residence was approximated as a rectangular prism with a roof geometry. Examples where geometrical approximations were implemented:

- The shape and extent of projection of bay windows were not recorded.
- Some roofs come down to different levels at different parts of the residence.
- The ground level changes around some residences, such as 45 St. Mary's Close in Hull where the ground level on the left side is 0.60 m higher than the ground level on the right side. The average of the two values was recorded as the house's relative height.

Figure 5.4.2.1: Geometrical Data



NOTE: Heights and widths were measured from the inside of the frame.

For each door on every fifth residence, the data in Table 5.4.2.3 were collected. Garage doors were not considered because the garage was considered to be outside the residence. As well, postal flaps were not always on a door, but were recorded as being affiliated with the closest door.

Table 5.4.2.3: Data Collected for Doors

Subclass	Fields	Notes	Importance
door's material	timber, glass, PVC	If glass, record material of frame.	Glass failure, flood rise rate, and other failure modes (locks and frames).
door's finish	painted, none obvious		
door's opening method	swing out swing in slide sideways slide up		
door's condition	no damage visible some damage unrepaired significant damage unrepaired	If damage, describe.	
door's geometrical properties	b, H, W, floor #, shape	To the nearest 0.1 m ± 0.1 m.	
if postal flap, properties	b, H, W, shape opens inwards/outwards	b and W to the nearest 0.1 m ± 0.1 m. Assume H = 0.05 m (based on field measurements) unless a greater height to the nearest 0.1 m could be measured.	Flood rise rate.
other openings	cat flap, window	If another type, describe.	Flood rise rate, glass failure, and other failure modes (locks, mullions, transoms, frames, screws).
if other openings, properties	b, H, W, opening method, shape	To the nearest 0.1 m ± 0.1 m.	

For each window on every tenth residence, the data in Table 5.4.2.4 were collected. Data on shutters would have been collected if any had been observed.

Table 5.4.2.4: Data Collected for Windows

Subclass	Fields	Notes	Importance
unit's geometrical properties	b, H, W, floor #, shape, critical location (yes/no) as defined by Approved Document N (1998) (see Chapter 10)	H and W to the nearest 0.25 m were recorded (see Chapter 10). b to the nearest 0.1 m ± 0.1 m was recorded.	Flood rise rate, glass failure, and other failure modes (locks, mullions, transoms, frames, screws)
unit's opening method	swings inwards, hinges in line on vertical axis swings outwards, hinges in line on vertical axis swings on central axis, hinges in line on vertical axis swings inwards, hinges in line on horizontal axis swings outwards, hinges in line on horizontal axis swings on central axis, hinges in line on horizontal axis slide vertical slide horizontal non-opening		
unit's manufactured properties	designed glass (yes/no) fake bars (yes/no) glazing (double/single)		
non-glass parts	aluminium, PVC, steel, timber		
non-glass parts' finish	painted, none obvious		
condition of unit and window	no damage visible some damage unrepaired significant damage unrepaired	If damage, describe.	

For each other opening of every fifth residence, the data in Table 5.4.2.5 were collected.

Table 5.4.2.5: Data Collected for Other Openings

Subclass	Fields	Notes	Importance
opening	air brick duct pipe vent	Notes were also made on weep holes, trickle vents, and openings for wires (e.g. telephone, cable, electricity).	Flood rise rate.
wall	front, back, left, right		
hole's geometrical properties	b, H, W	To the nearest 0.1 m ± 0.1 m. Estimates were made for dimensions smaller than 0.1 m.	

5.5 Survey Method: Notes and Definitions

5.5.1 House and Flat

If a residence has a separate residence above or below it, it is a flat. Otherwise, it is a house. Houses change to a new residence in the horizontal direction only while flats change to a new residence vertically always and often horizontally as well. The obvious expectation is that the number of ground floor flats should equal the number of first floor flats, but this situation does not occur because some flat complexes use the ground floor for non-residential purposes, such as garages, communal areas, and shops. In such instances, the residences were surveyed but the non-residential parts were not surveyed. Doors were attributed to blocks of flats rather than to a single residence.

Because ΔP from f_{diff} and v is influenced by the relative height of the residence, separating flats into individual residences is inappropriate. Instead, columns of flats were used for data which apply to the entire structure such as relative height. Flat columns can be effectively treated as a house for the approximations considered here. Due to differing configurations on each level, matching flats with flat columns was not always precise. Slight inconsistencies appear in the configuration tables of Chapter 6 which do not affect the overall results.

5.5.2 Exterior Walls

Residences have been assumed to be rectangles when viewed from above. Two octagonal buildings were seen on Canvey Island (7 Ferndale Crescent and 6 Haven Road) which were recorded as having eight exterior walls. Two cases of an L-shaped terraced middle house were seen in Hull (17 and 22 Mast Drive) which were recorded as 2opp.

Some residences were partially attached to their neighbours and had half-walls. The nearest whole number of walls was recorded. Some instances of a 3-storey house attached to a 2-storey house and a 2-storey house attached to a 1-storey house were observed. In such cases, a left or right wall appeared on only the uppermost storey. Flood vulnerability is focussed on lower storeys, hence for the purposes of flood vulnerability, this wall was not counted as an exterior wall.

Designating which wall is the front wall defines the back, left, and right walls, but is clearly an arbitrary designation. These labels are used solely to identify walls and are not taken to be significant otherwise.

One classification issue arose when two garages were attached in terms of deciding whether or not the wall between the garage and the residence should be considered to be an exterior wall. The garage-residence wall was assumed to be an exterior wall although any flood velocity component would be nearly eliminated before the flood water encounters this wall. If both garages are converted into an additional room, a rare situation, this model would be inaccurate, except that most garages do not extend the full length of the wall to which they are attached. Therefore, the residence could still be considered to have an exterior wall on the garage side.

The result is that residences with a garage attached to their neighbour would not be classified differently from residences for which the garage is not attached to their neighbour. The implication is that garages are considered to be instantaneously penetrable by flood water and that no difference is assumed between a garage-residence wall and a normal exterior wall. This approach applies when the garage is partially underneath the residence or built into the residence. This model of a residence is a valid first-order approximation for the geometry of a residence.

5.5.3 Storey or Floor Level

Storeys and floor levels were generally distinct. The few exceptions involve levels (identified mainly by windows) which were partway in between the distinct storeys/levels. When residence components appear halfway in between the main levels, the difference will appear in b. Flats were almost always on a single, clear storey. Some houses had different parts with different number of storeys, but the maximum number of storeys in the house has been recorded with a used loft counting as a full storey.

Storeys and floor levels were assumed to be evenly distributed over the entire level. Lofts did not always conform to such an assumption, but tended to cover the majority of the storey's plan area. Tiling may not have similar infiltration rates or strength characteristics to brickwork or blockwork, but this approximation is justifiable at the first order due to the small number of lofts present (Sections 6.1.3 and 6.2.3).

5.5.4 Relative Height of Residences

The measurement reported is height above Ordnance Datum Newlyn but the relative differences are important, not the absolute value. The data were obtained in March 1999 by the Environment Agency (England and Wales, U.K.) using LiDAR. Accuracy to the nearest 0.1 m ± 0.2 m is the best achievable (Brown, 2003). Some residences were built after March 1999, hence the topography's height was recorded for the relative height of the residence. The photographs of the residences were examined to ensure that the post-construction ground level does not differ from adjacent land.

5.5.5 Doors, Windows, and Other Openings

All doors, windows, and other openings were assumed to be vertical. Skylights would not be vertical, but were considered only with roof data.

Differentiating double-glazed and single-glazed units was based on expert judgement. Bermingham (2001) noted that double-glazed units have been installed in the U.K. since before 1970, but even in 1983 the target for commercial firms was predominantly upmarket residences. Using information from Chisholm (2001), Keiller (2001), and several commercial glazing companies in England (e.g. The Window Man, The BB Group, Hill Leigh Ltd., and Yehi), an approximate algorithm was developed for identifying glazing type. This algorithm was created with information from these sources, but the analysis and decisions may not necessarily be attributable directly to these sources.

- Single glazing:
  - -All units manufactured before ≈1990-1992.
  - -Most such units would be timber- or steel-framed.
  - -All single-glazed units were non-weather-stripped originally, although remedial action may have added weather-stripping.
- Double glazing:
  - -All units manufactured after ≈1990-1992.
  - -Most such units would be PVC- or aluminium-framed.
  - -Timber-framed units could occur commonly, but steel-framed units would be rare.
  - -All double-glazed units are weather-stripped, but the weather-stripping may have become damaged and have reduced effectiveness.

Identifying the age of units is clearly challenging, but various clues assisted in identifying glazing type. Residences built after 1992 likely had double-glazed units. Units in residences built before 1992 were examined more closely, but frame condition and type facilitated the identification. Consulting experts (e.g. Baggs, 2001; Campbell, 2001-2; McEvoy, 2001-2) and closely inspecting some residences reduced ambiguities to just a few residences.

5.5.6 Damage Observed

Any observed damage to residence components had to be obvious to be entered as "some damage visible". Labels of "significant damage visible" implies severe problems.

Obvious repairs are not included because analysing the age and effectiveness of the repair is not overly relevant and is subject to major errors. Repair may make the structure stronger than it was originally or may have little effect, but all residences have non-visible weaknesses and a repaired structure is not different from an undamaged one in this respect. Identifying and classifying damage is important because it indicates:

- The distribution and types of pre-existing damage modes thereby indicating possible ways in which flood actions would affect the structure.
- Mitigation and prevention strategies such as insurance companies providing incentives for keeping a structure in repair, i.e. using maintenance as a mitigation strategy.

5.6 Conclusions

This Chapter has developed a residence survey method to be applied in the case study sites for determining the first-order physical vulnerability of residences to ΔP from f_{diff} and v. A new definition of residence has been suggested for flood vulnerability studies and the data to be collected have been identified, categorised, and defined. Chapter 6 describes the results and analysis from the survey.

6. Residence Survey Results

The ± values reported are standard deviations unless otherwise stated. Standard deviation is reported because the sample size was so large that measurement and data errors became negligible and would not provide a good indication of the distributions' spreads.

A parameter's typical value is given by the median rather than the arithmetic mean (Ross, 2000; Walpole *et al.*, 1998). In choosing representative parameters, more weight is given to the median than to the mean.

Percentages may not add up to 100% due to rounding.

Much of the data collected, and all principal observations, have been summarised in this Chapter. Specific details or results not reported are referred to and used in other Chapters.

6.1 Observations: Canvey Island

6.1.1 Configuration: Photographic Survey

Table 6.1.1.1 categorises 417 residences over 21 streets which were photographed. Not all streets were covered completely. The percentages for flats are calculated using the sum of houses and flats. The percentages for flat columns are calculated using the sum of houses and flat columns. The first percentages for houses are calculated using the sum of houses and flats; the second percentages use the sum of houses and flat columns.

Table 6.1.1.1: Canvey Island Photographic Survey

Configuration for Houses	j →	1	2	3
	w1	--	6 (1%, 2%)	--
	w2adj	--	8 (2%, 2%)	--
	w2opp	6 (1%, 2%)	24 (6%, 7%)	--
	w3	22 (5%, 6%)	55 (13%, 16%)	--
	w4	58 (14%, 17%)	93 (22%, 27%)	7 (2% 2%)
Configuration for Flats Using Level	Level →	ground	1st floor	2nd floor
	w1	3 (1%)	3 (1%)	3 (1%)
	w2adj	23 (6%)	25 (6%)	4 (1%)
	w2opp	10 (2%)	11 (3%)	2 (0%)
	w3	24 (6%)	24 (6%)	4 (1%)
	w4	1 (0%)	1 (0%)	--
Configuration for Flat Columns Using j	j →		2	3
	w1		--	3 (1%)
	w2adj		21 (6%)	4 (1%)
	w2opp		9 (3%)	2 (1%)
	w3		20 (6%)	4 (1%)
	w4		1 (0%)	--

6.1.2 Configuration: Quick Survey

Table 6.1.2.1 categorises 4,872 residences out of approximately 20,000 residences on Canvey Island which were covered by the quick survey. The percentages for flats are calculated using the sum of houses and flats. The percentages for flat columns are calculated using the sum of houses and flat columns. The first percentages for houses are calculated using the sum of houses and flats; the second percentages use the sum of houses and flat columns.

Table 6.1.2.1: Canvey Island Quick Survey

Configuration for Houses	j →	1	2	3
	w1	--	7 (0%, 0%)	--
	w2adj	4 (0%, 0%)	28 (1%, 1%)	--
	w2opp	13 (0%, 0%)	370 (8%, 8%)	26 (1%, 1%)
	w3	327 (7%, 7%)	1,097 (23%, 24%)	16 (0%, 0%)
	w4	976 (20%, 21%)	1,533 (31%, 33%)	30 (1%, 1%)
	8 exterior walls	--	2 (0%)	--
Configuration for Flats Using Level	Level →	ground	1st floor	2nd floor
	w1	26 (1%)	28 (1%)	3 (0%)
	w2adj	45 (1%)	41 (1%)	8 (0%)
	w2opp	25 (1%)	40 (1%)	13 (0%)
	w3	99 (2%)	99 (2%)	10 (0%)
	w4	3 (0%)	3 (0%)	--
Configuration for Flat Columns Using j	j →		2	3
	w1		25 (1%)	3 (0%)
	w2adj		35 (1%)	8 (0%)
	w2opp		25 (1%)	13 (0%)
	w3		89 (2%)	10 (0%)
	w4		3 (0%)	--
Caravans/mobiles were being removed to be replaced by new developments.				

The photographic survey is a reasonable representation of all residences.

6.1.3 Storeys

Houses (4,429 observed, 91% of residences):

- 1 storey 1,320 (30%)
- 2 storey 3,037 (69%)
- 3 storey 72 (2%)

Flats (443 observed, 9% of residences):

- ground floor 198 (45%)
- 1st floor 211 (48%)
- 2nd floor 34 (8%)

No residences have basements.

Loft Used (out of 341 non-flat roofs):

- no 319 (94%)
- yes 22 (6%)

Approximating a loft as a storey is justified, since relatively few residences had lofts and some of these lofts were nearly indistinguishable from storeys.

6.1.4 Relative Height

1,536 houses and flat columns over all quick survey areas were examined (Table 6.1.4.1). A comparison was made with the statistics for all data points over Canvey Island. This comparison must be viewed with caution because:

- For each residence, only one point was recorded to be representative of the residence while the full data set involves many points per residence.
- The full data set includes all areas of Canvey Island, not just the residential sectors.

Table 6.1.4.1: Relative Height of Canvey Island Residences

Height (m)	Quick Survey Data		Full Canvey Data	
	Number	%	Number	%
1.0	--	--	860	0%
1.1	--	--	1,609	1%
1.2	--	--	3,635	1%
1.3	7	0%	7,832	3%
1.4	23	1%	18,855	6%
1.5	139	9%	43,776	14%
1.6	453	29%	77,952	26%
1.7	457	30%	78,548	26%
1.8	124	8%	21,247	7%
1.9	90	6%	11,248	4%
2.0	98	6%	10,983	4%
2.1	78	5%	12,359	4%
2.2	50	3%	10,318	3%
2.3	17	1%	4,695	2%
2.4	--	--	540	0%
Total	1,536		304,457	

The quick survey data have a mean of 1.7 m ± 0.2 m with a median of 1.7 m.
The full Canvey Island data have a mean of 1.7 m ± 0.2 m with a median of 1.6 m.

Table 6.1.4.1 indicates that the sample chosen for the quick survey is representative of Canvey Island. Discrepancies in percentages are explained by the cautions listed above. A

potential trend is that residences do not appear in the lowest 2% of height categories. Either houses and flats have not been built over the lowest part of Canvey Island or the ground level of the lowest residential communities was artificially raised before residence construction. 89% of residences lie within 0.3 m of the 1.7 m mark. For a typical residence, 1.7 m will be used.

6.1.5 Ground to Ground Floor Bottom

Out of 341 houses and flat columns in the photographic survey:

- 0.1 m 1 (0%)
- 0.2 m 94 (28%)
- 0.3 m 179 (52%)
- 0.4 m 46 (13%)
- 0.5 m 21 (6%)

4 and 6 George Close are not included because they were set back behind gates and could not be viewed properly. The mean is 0.30 m ± 0.08 m and the median is 0.3 m. For a typical residence, 0.3 m will be used.

6.1.6 Wall Height

Heights for the distance from ground floor to roof bottom appear in Table 6.1.6.1 for 343 houses and flat columns in the photographic survey. The mean height of a house storey and flat column level can be calculated assuming:

- Storeys and levels are evenly distributed over the measured height from ground floor to roof bottom.
- Residences which appear too short (shaded in grey in the table) have a low-hanging roof and the height reported does not include the top storey/level. This situation was confirmed by re-examining the photographs.

Three ranges are delineated by double lines in the table.

Table 6.1.6.1: Storey/Level Height for Canvey Island

Value	Storey/Level Height	Number with Specific Configuration Houses (j)			Flats (j)	
		1	2	3	2	3
2.0 m	2.0 m	3	1	--	--	--
2.1 m	2.1 m	45	9	--	--	--
2.2 m	2.2 m	22	9	--	--	--
2.3 m	2.3 m	5	--	--	--	--
2.4 m	2.4 m	2	3	--	--	--
2.5 m	2.5 m	9	--	--	--	--
2.6 m	2.6 m	--	1	--	--	--
4.5 m	2.2 m (2.25 m)	--	2	--	--	--
4.6 m	2.3 m	--	15	--	--	--
4.7 m	2.4 m (2.35 m)	--	62	--	--	--
4.8 m	2.4 m	--	54	1	3	--
4.9 m	2.4 m (2.45 m)	--	26	--	26	--
5.0 m	2.5 m	--	3	--	18	--
5.1 m	2.6 m (2.55 m)	--	1	--	--	--
5.2 m	2.6 m	--	--	--	4	5
7.3 m	2.4 m	--	--	6	--	--
7.7 m	2.6 m	--	--	--	--	6
8.5 m	2.8 m	--	--	--	--	2
Mean Storey/Level Height		2.2 m	2.4 m	2.4 m	2.5 m	2.6 m
Median Storey/Level Height		2.1 m	2.35 m	2.4 m	2.45 m	2.6 m

For all residences, the mean is 2.34 m ± 0.15 m and the median is 2.4 m. 2.4 m is used to represent all residences.

6.1.7 Exterior Perimeter

Table 6.1.7.1: Residence Π for Canvey Island

	j →	1	2	3
Houses	w1	--	6.1 m ± 0.6 m	--
	w2adj	--	12.4 m ± 2.9 m	--
	w2opp	16.4 m ± 0.2 m	11.8 m ± 0.5 m	--
	w3	26.9 m ± 4.9 m	23.1 m ± 2.3 m	--
	w4	37.5 m ± 7.4 m	40.0 m ± 8.3 m	45.0 m ± 20.7 m
Flat Columns	j →		2	3
	w1		--	16.8 m ± 3.2 m
	w2adj		18.9 m ± 1.4 m	16.3 m ± 3.7 m
	w2opp		15.0 m ± 4.5 m	18.3 m ± 0.0 m
	w3		24.6 m ± 3.3 m	25.2 m ± 2.7 m
	w4		38.9 m (sample size 1)	--

No patterns emerge from Table 6.1.7.1 except the general observation that Π tends to increase as the number of walls increases.

The 271 houses have a mean of 30.5 m ± 12.7 m and a median of 31.0 m (data were not available for eight houses on George Close).
The 64 flat columns have a mean of 20.7 m ± 5.2 m and a median of 20.2 m.
The 335 houses and flat columns have a mean of 28.7 m ± 12.3 m and a median of 26.5 m.
Since the means are close with respect to the standard deviation, houses and flat columns may be combined:

- 0.0 m ≤ Π ≤ 9.9 m: 6
- 10.0 m ≤ Π ≤ 19.9 m: 69
- 20.0 m ≤ Π ≤ 29.9 m: 116
- 30.0 m ≤ Π ≤ 39.9 m: 85
- 40.0 m ≤ Π ≤ 49.9 m: 49
- 50.0 m ≤ Π ≤ 59.9 m: 5
- 60.0 m ≤ Π ≤ 69.9 m: 3
- 70.0 m ≤ Π ≤ 79.9 m: 1
- 80.0 m ≤ Π ≤ 89.9 m: 0
- 90.0 m ≤ Π ≤ 99.9 m: 1

6.1.8 Plan Area

Table 6.1.8.1: Residence A for Canvey Island

	j →	1	2	3
Houses	w1	--	34.1 m^2 ± 0.6 m^2	--
	w2adj	--	36.1 m^2 ± 4.3 m^2	--
	w2opp	63.3 m^2 ± 0.7 m^2	47.8 m^2 ± 6.0 m^2	--
	w3	62.4 m^2 ± 7.9 m^2	49.5 m^2 ± 10.2 m^2	--
	w4	66.8 m^2 ± 23.9 m^2	72.3 m^2 ± 29.0 m^2	85.8 m^2 ± 56.3 m^2
Flat Columns	j →		2	3
	w1		--	67.8 m^2 ± 24.8 m^2
	w2adj		56.3 m^2 ± 6.9 m^2	55.4 m^2 ± 0.0 m^2
	w2opp		59.9 m^2 ± 9.9 m^2	105 m^2 ± 0 m^2
	w3		59.6 m^2 ± 7.6 m^2	82.4 m^2 ± 26.0 m^2
	w4		72.0 m^2 (sample size 1)	--

No patterns emerge from Table 6.1.8.1.

Out of 271 houses, the mean is 61.6 m^2 ± 25.0 m^2 and the median is 57.5 m^2 (data were not available for eight houses on George Close).
Out of 64 flat columns, the mean is 61.9 m^2 ± 14.5 m^2 and the median is 56.6 m^2.
Out of 335 houses and flat columns, the mean is 61.7 m^2 ± 23.3 m^2 and the median is 57.2 m^2.

Since the means are close, especially with respect to the standard deviation, houses and flats may be combined:

- 0.0 m^2 ≤ A ≤ 24.9 m^2: 0
- 25.0 m^2 ≤ A ≤ 49.9 m^2: 91
- 50.0 m^2 ≤ A ≤ 74.9 m^2: 189
- 75.0 m^2 ≤ A ≤ 99.9 m^2: 35
- 100.0 m^2 ≤ A ≤ 124.9 m^2: 14
- 125.0 m^2 ≤ A ≤ 149.9 m^2: 3
- 150.0 m^2 ≤ A ≤ 174.9 m^2: 1
- 175.0 m^2 ≤ A ≤ 199.9 m^2: 0
- 200.0 m^2 ≤ A ≤ 224.9 m^2: 2

6.1.9 Design Purpose

All residences were constructed as residential properties, including the two houses with eight exterior walls. For 6 Haven Road (Dutch Cottage, built 1621) see Pevsner (1954) and for 7 Ferndale Crescent (Dutch Cottage imitation, built 1945-1969) see the OS Historical Maps (various dates). The other surviving Dutch Cottage on Canvey Island (on Canvey Road just south of Hill Hall, built in 1618) fell within the area of the quick survey, but is currently a museum. Since it is not currently a residential property, it was not surveyed. All residences were built for residential purposes, hence plenty opportunity exists for reducing flood vulnerability during construction.

6.1.10 Age

More than two-thirds of the residences were built between 1960 and 1978, but they are rapidly being knocked down to be replaced by modern residences. For the quick survey residences:

- 0% are pre-1923
- < 10% are 1923-1960
- > 90% are post-1960

6.1.11 Walls

1,245 walls were observed.

Material and Cavities:

- All residences observed had cavity walls with a brickwork exterior except one residence (4 George Close) for which all four walls were a combination of brickwork and blockwork on the exterior.
- 136 (11%) walls had sections covered by tile, timber siding (also termed facing, cladding, battens, batts, boards, weatherboarding, or sheeting), or plastic siding. Some walls clearly had brickwork behind this finish while others were indeterminable.

Assuming that all residence walls are cavity walls with a brickwork external leaf and a blockwork internal leaf is reasonable.

Finish:

• none	74%
• pebble dash	1%
• render	7%
• stone facing	0% (one wall)
• combination	17%, most commonly:
-none and render	5%
-none and tile	5%

Damage:

- no damage visible 97%
- some damage visible 2%
  - -6 had brick damage
  - -22 had finish damage
  - -3 had water marks or stains.
- significant damage visible 0% (three walls)
  - -All three had finish damage.

Assuming that all residence walls are in good condition is reasonable.

6.1.12 Doors

140 doors were observed. Almost all front doors had H = 2.0 m and W = 0.8 m. The opening method for most was swinging inwards, but 4% swing outwards and one front door slides sideways. For material:

- glass with metal frame: 3%
- glass with PVC frame: 23%
- glass with timber frame: 12%
- PVC: 23%
- timber: 40%

44% of houses and flat columns also had a back door which slides sideways and which was made in two glass units, each with H = 2.0 m and 0.8 m ≤ W ≤ 1.1 m. A few exceptions with smaller W or different opening styles exist. Side doors and non-sliding back doors were similar to the front doors. On the ground floors, b for doors was effectively the same as b for the residence's ground floor.

Other principal observations:

- Less than 2% of doors displayed damage and none had significant damage.
- 44% of front doors had a postal flap, usually with b ≈ 1 m. Few other doors had a postal flap.
- Small glass units (H and W ≤ 0.25 m) were common.
- No cat flaps were observed.

6.1.13 Windows

1,062 window units were observed. 1.5% had H or W > 1.0 m and 37% had both H and W ≤ 0.50 m. Materials for the units' frame, transoms, and mullions were:

- metal: 3%
- PVC: 55%
- timber: 41%

Other principal observations:

- 51% of units did not open. 98% of opening units opened by swinging outwards.
- Four units were not rectangular.
- Six units displayed minor damage with the rest having no damage.
- 90% of units were double-glazed.
- 13% were not simple, flat plates but had an additional design in the glass.
- One house and three flat columns had large window units with b ≤ 0.5 m. Other window units either had one dimension ≤ 0.25 m or b ≥ 0.8 m.

6.1.14 Other Openings

Other than air bricks, no large openings were observed with b < 1.5 m. The only large openings with 1.5 m ≤ b < 2.0 were two vents and two window fans. Pipes, ducts, and inlets for cables were small, with R ≤ 0.1 m^2. Air bricks, most commonly with H = 0.08 m and W = 0.2 m, were placed every 1-2 m resulting in an average of 3-5 air bricks per wall. The air bricks' b = 0.17 m ± 0.05 m for the common type. Fifteen large air bricks with H = W = 0.2 m were observed over five residences on two streets, but b = 1.5 m ± 0.8 m for them.

6.1.15 Preliminary Comments on Canvey Island

Canvey Island is a mishmash of inconsistent, haphazard, and ongoing development. The high proportion of bungalows (30% of houses) induces high flood vulnerability in the population, particularly considering that many bungalow residents would be pensioners living on their own. Residents in ground floor flats (45% of flats) could seek refuge in someone else's flat if they are given adequate warning, but would still incur high losses and would not be protected from residence destruction.

A modern residential home (Silverpoint Court, built in 1995) sits immediately behind the sea wall at the eastern end of Canvey Island. As noted in Section 4.2, evacuating Canvey Island by land presents difficulties. The high point of Canvey Island, the landfill in the northeast, could house an emergency shelter, but has only one access road.

A few of the older residences still have their original windows and doors (e.g. 39 Cleveland Road) which display marked deterioration. The salt from the North Sea wind causes considerable damage to some residence components (e.g. the metal frames on the windows of 302 Furtherwick Road).

Residence age is not always correlated with natural disaster vulnerability, due to differing maintenance practices. For example:

- The oldest building viewed, the Dutch Cottage from 1621, appeared to be in better overall condition than many buildings from the 1970s.
- At the time it was surveyed, the left wall of 8 Vicarage Close was being replaced entirely. The house was built before 1960, but one-third of its walls would likely be pristine during part of 2001.
- Western Esplanade has two sets of adjacent apartment blocks built after 1980, Kent View and Thames Court. The latter is in better condition than the former.

The decision to focus on damage rather than age to determine residence condition (Section 5.3) has been justified.

6.2 Observations: Hull

6.2.1 Configuration: Photographic Survey

Table 6.2.1.1 categorises 646 residences over 19 streets which were photographed. Not all streets were covered completely. The percentages for flats are calculated using the sum of houses and flats. The percentages for flat columns are calculated using the sum of houses and flat columns. The first percentages for houses are calculated using the sum of houses and flats; the second percentages use the sum of houses and flat columns.

Table 6.2.1.1: Hull Photographic Survey

Configuration for Houses	j →	1	2	3	
	w1	--	--	--	
	w2adj	--	4 (1%, 1%)	--	
	w2opp	3 (0%, 1%)	28 (4%, 6%)	8 (1%, 2%)	
	w3	18 (3%, 4%)	178 (27%, 37%)	12 (2%, 2%)	
	w4	1 (0%, 0%)	139 (21%, 29%)	--	
	terraced middle but L-shaped	2 (0%, 0%)	--	--	
Configuration for Flats Using Level	Level →	ground	1st floor	2nd floor	3rd floor
	w1	21 (3%)	20 (3%)	6 (1%)	4 (1%)
	w2adj	2 (0%)	2 (0%)	5 (1%)	--
	w2opp	31 (5%)	31 (5%)	19 (3%)	2 (0%)
	w3	36 (6%)	37 (6%)	33 (5%)	4 (1%)
	w4	--	--	--	--
Configuration for Flat Columns Using j	j →		2	3	4
	w1		10 (2%)	4 (1%)	4 (1%)
	w2adj		1 (0%)	5 (1%)	--
	w2opp		12 (2%)	17 (4%)	2 (0%)
	w3		4 (1%)	29 (6%)	4 (1%)
	w4		--	--	--

6.2.2 Configuration: Quick Survey

Table 6.2.2.1 categorises all residences in the identified communities of Hull, i.e. 100% coverage totalling 1,769 residences. The percentages for flats are calculated using the sum of houses and flats. The percentages for flat columns are calculated using the sum of houses and flat columns. The first percentages for houses are calculated using the sum of houses and flats; the second percentage use the sum of houses and flat columns.

Table 6.2.2.1: Hull Quick Survey

Configuration for Houses	j →	1	2	3	
	w1	--	--	--	
	w2adj	--	10 (1%, 1%)	--	
	w2opp	3 (0%, 0%)	115 (7%, 8%)	14 (1%, 1%)	
	w3	25 (1%, 2%)	604 (34%, 44%)	17 (1%, 1%)	
	w4	17 (1%, 1%)	362 (20%, 26%)	--	
	terraced middle but L-shaped	2 (0%, 0%)	--	--	
Configuration for Flats Using Level	Level →	ground	1st floor	2nd floor	3rd floor
	w1	21 (1%)	23 (1%)	9 (1%)	6 (0%)
	w2adj	15 (1%)	20 (1%)	24 (1%)	9 (1%)
	w2opp	96 (5%)	100 (6%)	78 (4%)	4 (0%)
	w3	63 (4%)	65 (4%)	56 (3%)	8 (0%)
	w4	--	--	--	--
Configuration for Flat Columns Using j	j →		2	3	4
	w1		10 (1%)	5 (0%)	6 (0%)
	w2adj		--	14 (1%)	9 (1%)
	w2opp		22 (2%)	73 (5%)	4 (0%)
	w3		9 (1%)	50 (4%)	8 (1%)
	w4		--	--	--
caravan/mobile (all are 1 storey, 4 exterior walls)			3 (0%)		

The photographic survey is a reasonable representation of all residences.

6.2.3 Storeys

Houses (1,169 observed, 66% of residences):

- 1 storey 47 (4%)
- 2 storey 1091 (93%)
- 3 storey 31 (3%)

Flats (597 observed, 34% of residences)

- ground floor 195 (33%)
- 1st floor 208 (35%)
- 2nd floor 167 (28%)
- 3rd floor 27 (5%)

No residences have basements.

Loft Used (out of 481 non-flat roofs):

- no 441 (92%)

- yes 40 (8%)

Approximating a loft as a storey is justified, since relatively few residences had lofts and some of these lofts were nearly indistinguishable from storeys.

6.2.4 Relative Height

A comparison was not made with the statistics for all data points over the quick survey areas of Hull because 100% of residences in the most vulnerable areas of Hull were covered by the quick survey. For relative height, all residences on at least every second street were recorded, totalling 893 houses and flat columns and including all residences in Hessle Foreshore.

Table 6.2.4.1: Relative Height of Hessle Foreshore Residences

Height (m)	Quick Survey Data	
	Number	%
5.1	27	27%
5.2	0	0%
5.3	2	2%
5.4	9	9%
5.5	9	9%
5.6	0	0%
5.7	10	10%
5.8	0	0%
5.9	0	0%
6.0	3	3%
6.1	0	0%
6.2	0	0%
6.3	0	0%
6.4	0	0%
> 6.5, < 10.0	39	39%
Total	99	

The Quick Survey Data below 6.5 m have an mean of 5.4 m ± 0.3 m, a range which encompasses 95% of residences below 6.5 m, and a median of 5.4 m. This community is reasonably flat at the shoreline but then rises up a steep slope. When considering flood scenarios, Hessle is thus divided into these two subcommunities: one with a relative height of 5.4 m and one with a relative height of approximately 7 m.

Table 6.2.4.2: Relative Height of Victoria Dock Village Residences

Height (m)	Quick Survey Data	
	Number	%
4.3	1	0%
4.4	0	0%
4.5	0	0%
4.6	14	2%
4.7	39	6%
4.8	72	11%
4.9	79	12%
5.0	56	8%
5.1	31	5%
5.2	39	6%
5.3	32	5%
5.4	27	4%
5.5	44	6%
5.6	81	12%
5.7	35	5%
5.8	13	2%
5.9	8	1%
6.0	3	0%
6.1	3	0%
6.2	0	0%
6.3	29	4%
6.4	13	2%
6.5	9	1%
6.6	13	2%
6.7	15	2%
6.8	13	2%
6.9	4	1%
Total	678	

The Quick Survey Data have an mean of 5.4 m ± 0.6 m and a median of 5.3 m. Higher residences occurred towards the western end and southeast corner with coastal residences slightly higher than non-coastal residences (Figure 6.2.4.1). Victoria Dock Village is reasonably flat.

Figure 6.2.4.1: Relative Height of Victoria Dock Village

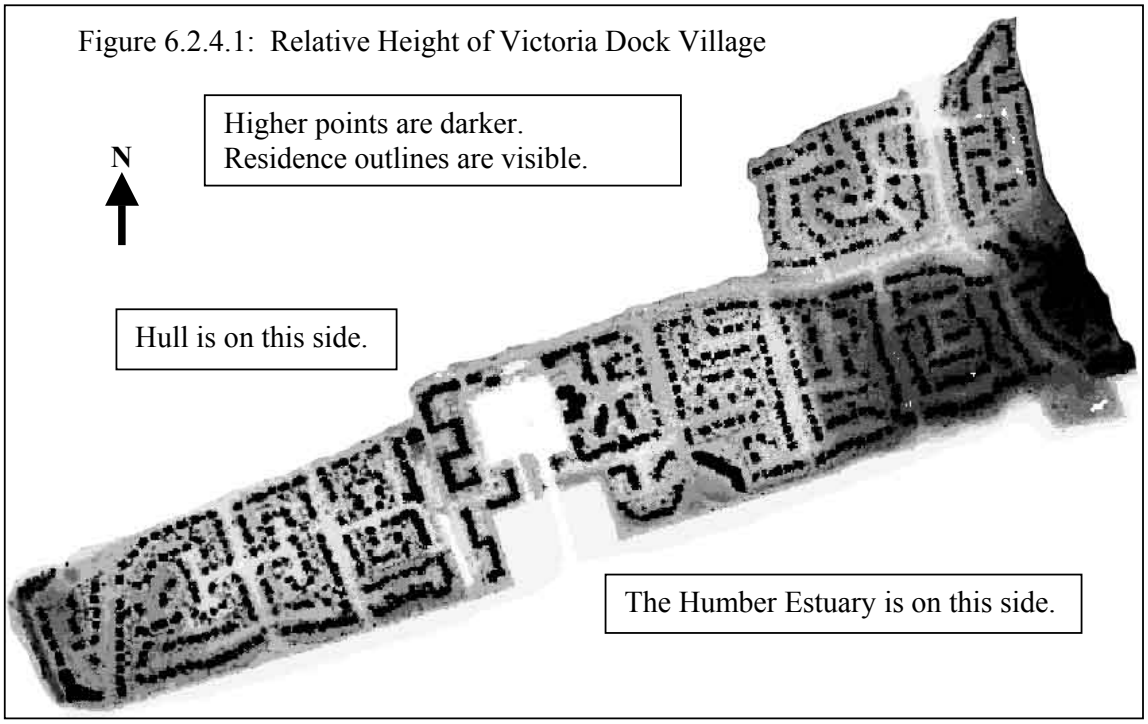


Table 6.2.4.3: Relative Height of Wellington Street Area Residences

Height (m)	Quick Survey Data	
	Number	%
4.1	4	3%
4.2	6	5%
4.3	7	6%
4.4	19	16%
4.5	32	28%
4.6	31	27%
4.7	15	13%
4.8	1	1%
4.9	1	1%
Total	116	

The Quick Survey Data have an mean of 4.5 m ± 0.2 m and a median of 4.5 m. 98% of residences lie within 0.3 m of the 4.4 m mark. Wellington Street may be considered a single community with a relative height of 4.5 m.

6.2.5 Ground to Ground Floor Bottom

Out of 483 houses and flat columns in the photographic survey:

- 0.2 m 451 (93%)
- 0.3 m 25 (5%)
- 0.6 m 6 (1%) All six residences of Riverside Court.

- 0.7 m 1 (0%) 20 St. Mary's Close

The mean is 0.21 m ± 0.05 m and the median is 0.2 m. For a typical residence, 0.2 m will be used. While the variations up to 0.5 m impact flood vulnerability, they are rare. Since a few residences are higher than others, it appears that raising all residences could be completed if desired.

6.2.6 Wall Height

Heights for the distance from ground floor to roof bottom appear in Table 6.2.6.1 for 483 houses and flat columns in the photographic survey. The mean height of a house storey and flat column level can be calculated assuming that storeys and levels are evenly distributed over the measured height from ground floor to roof bottom. Four ranges are delineated by double lines in the table.

Table 6.2.6.1: Storey/Level Height for Hull

Value	Storey/Level Height	Number with Specific Configuration					
		Houses (j)			Flats (j)		
		1	2	3	2	3	4
2.1 m	2.1 m	24	--	--	--	--	--
4.3 m	2.2 m (2.15 m)	--	2	--	--	--	--
4.4 m	2.2 m	--	8	--	--	--	--
4.6 m	2.3 m	--	1	--	--	--	--
4.7 m	2.4 m (2.35 m)	--	14	--	16	--	--
4.8 m	2.4 m	--	196	--	--	--	--
4.9 m	2.4 m (2.45 m)	--	83	--	--	--	--
5.0 m	2.5 m	--	36	--	11	--	--
5.1 m	2.6 m (2.55 m)	--	8	--	--	--	--
5.8 m	2.9 m	--	1	--	--	--	--
7.0 m	2.3 m	--	--	--	--	6	--
7.3 m	2.4 m	--	--	--	--	8	--
7.6 m	2.5 m	--	--	14	--	20	--
7.8 m	2.6 m	--	--	--	--	8	--
8.0 m	2.7 m	--	--	6	--	10	--
8.1 m	2.7 m	--	--	--	--	1	--
10.1 m	2.5 m	--	--	--	--	--	2
10.2 m	2.6 m (2.55 m)	--	--	--	--	--	4
11.6 m	2.9 m	--	--	--	--	--	4
Mean Storey/Level Height		2.1	2.4	2.6	2.4	2.5	2.7
Median Storey/Level Height		2.1	2.4	2.5	2.2	2.5	2.6

For all residences, the mean is 2.4 m ± 0.1 m and the median is 2.4 m. 2.4 m is used to represent all residences.

6.2.7 Exterior Perimeter

Table 6.2.7.1: Residence Π for Hull

	j →	1	2	3
Houses	w1	--	--	--
	w2adj	--	14.8 m ± 4.6 m	--
	w2opp	17.8 m ± 0.0 m	10.7 m ± 2.1 m	10.1 m ± 0.0 m
	w3	24.8 m ± 0.6 m	18.3 m ± 2.5 m	18.1 m ± 0.0 m
	w4	32.4 m (sample size 1)	35.1 m ± 4.0 m	--
	j →	2	3	4
Flat Columns	w1	5.0 m ± 0.0 m	17.1 m ± 0.0 m	17.1 m ± 0.0 m
	w2adj	7.0 m (sample size 1)	25.0 m (sample size 1)	--
	w2opp	25.2 m ± 7.2 m	23.6 m ± 5.5 m	24 m ± 0.0 m
	w3	26.6 m ± 4.1 m	31.5 m ± 7.2 m	24.8 m ± 0.0 m
	w4	--	--	--

No patterns emerge from Table 6.2.7.1 except the general observation that Π tends to increase as the number of walls increases.

The 393 houses have a mean of 23.9 m ± 9.3 m and a median of 20.8 m.
The 90 flat columns have a mean of 23.4 m ± 9.8 m and a median of 24.3 m.
The 483 houses and flat columns have a mean of 23.8 m ± 9.4 m and a median of 21.0 m.

Since the means are close, especially with respect to the standard deviation, houses and flat columns may be combined:

- 0.0 m ≤ Π ≤ 9.9 m: 28
- 10.0 m ≤ Π ≤ 19.9 m: 184
- 20.0 m ≤ Π ≤ 29.9 m: 108
- 30.0 m ≤ Π ≤ 39.9 m: 152
- 40.0 m ≤ Π ≤ 49.9 m: 9
- 50.0 m ≤ Π ≤ 59.9 m: 0
- 60.0 m ≤ Π ≤ 69.9 m: 1
- 70.0 m ≤ Π ≤ 79.9 m: 1

6.2.8 Plan Area

Table 6.2.8.1: Residence A for Hull

	j →	1	2	3
Houses	w1	--	--	--
	w2adj	--	42.7 m^2 ± 7.9 m^2	--
	w2opp	65.0 m^2 ± 0.0 m^2	35.2 m^2 ± 4.9 m^2	39.7 m^2 ± 0.0 m^2
	w3	65.0 m^2 ± 0.0 m^2	37.6 m^2 ± 7.2 m^2	39.7 m^2 ± 0.0 m^2
	w4	65.0 m^2 (sample size 1)	61.7 m^2 ± 16.5 m^2	--
	j →	2	3	4
Flat Columns	w1	41.6 m^2 ± 0.0 m^2	91.8 m^2 ± 0.0 m^2	91.8 m^2 ± 0.0 m^2
	w2adj	41.6 m^2 (sample size 1)	70.2 m^2 (sample size 1)	--
	w2opp	56.7 m^2 ± 4.5 m^2	68.0 m^2 ± 9.3 m^2	24.0 m^2 ± 0.0 m^2
	w3	58.9 m^2 ± 3.3 m^2	71.0 m^2 ± 24.6 m^2	24.8 m^2 ± 0.0 m^2
	w4	--	--	--

No patterns emerge from Table 6.2.8.1.

The 393 houses have a mean of 48.0 m^2 ± 16.3 m^2 and the median is 42.8 m^2.
The 90 flat columns have a mean of 67.1 m^2 ± 19.6 m^2 and the median is 62.4 m^2.
The 483 houses and flat columns have a mean of 51.5 m^2 ± 18.5m^2 and the median is 44.7 m^2.

Since the mean for houses and flat columns is close to the mean for each separate category with respect to the standard deviation, houses and flat columns may be combined:

- 0.0 m^2 ≤ A ≤ 24.9 m^2: 2
- 25.0 m^2 ≤ A ≤ 49.9 m^2: 256
- 50.0 m^2 ≤ A ≤ 74.9 m^2: 182
- 75.0 m^2 ≤ A ≤ 99.9 m^2: 37
- 100.0 m^2 ≤ A ≤ 124.9 m^2: 4
- 125.0 m^2 ≤ A ≤ 149.9 m^2: 0
- 150.0 m^2 ≤ A ≤ 174.9 m^2: 1
- 175.0 m^2 ≤ A ≤ 199.9 m^2: 1

6.2.9 Design Purpose

The residences' design purpose was:

- 1,739 (98%) were constructed as residences.
- 5 (0%), in one block of flats (Minerva View), were first a customs house and then a museum before being converted to residences after 1991.
- 9 (1%), in one block of houses (Belmont Farm 1,2,3,4,5,6,7,9,10), appear to have been industrial or possibly commercial, but were not converted to residences until after 1990. The most likely use would be boat houses.
- 11 (1%), the oldest buildings on Cliff Road (Cliff Cottages (all five), Riverside, Riverside Cottage, Riverside Cottages (all three), and Wintersgill) were likely residential from the start but it is difficult to confirm. Cliff Cottages 1,2,3,4 are old Coast Guard cottages, presumably used by rescuers to live, either constantly or while on shift.
- 3 (0%) are caravans/mobiles.

Since most residences were built for residential purposes, plenty opportunity exists for reducing flood vulnerability during construction.

6.2.10 Age

For the quick survey residences:

- pre-1910 19 (1%)

  These properties are in Hessle Foreshore and were built on the higher parts of the coast. The other Hessle Foreshore properties, built after 1990, make use of other land, both on the lower part of the coast and moving up Hessle Cliff.
- 1910-1928 5 (0%) but 1 building (Minerva View Flats)
- 1927-1952 1 (0%) (Cliff Cottage 5)
- 1980s 258 (15%) (all residences in the Wellington Street Area; no others)
- after 1990 1,484 (84%)

6.2.11 Walls

1,837 walls were observed.

Material:

- 94% had a brickwork exterior
- 6% had a combination of brickwork and blockwork on the exterior

Cavities:

- cavity 96%
- non-cavity 2% (16 residences)
- combination 1% (6 residences: Belmont Farm 1,5,6,7,8,9)

Assuming that all residence walls are cavity walls with a brickwork external leaf and a blockwork internal leaf is reasonable.

Finish:

- none 89%
- paint 1%
- render 1%
- none & render 7%
- none & paint 2%

Damage:

- no damage visible 98%
- some damage visible 2% (29 walls)
  - -3 had brick damage
  - -1 had erosion
  - -4 had finish damage
  - -22 had water marks or stains
- significant damage visible 0% (one wall)
  - -Had severe water marks and stains.

Assuming that all residence walls are in good condition is reasonable.

6.2.12 Doors

272 doors were observed. Almost all front doors had H = 2.0 m and W = 0.8 m. The opening method for most was swinging inwards, but 3% swing outwards and one front door slides sideways. For material:

- glass with PVC frame: 43%
- glass with timber frame: 5%
- PVC: 29%
- timber: 62%

95% of houses and flat columns also had a back door which slides sideways and which was made in two glass units, each with H = 2.0 m and 0.6 m ≤ W ≤ 1.1 m. Side doors and non-sliding back doors were similar to the front doors. On the ground floors, b for doors was effectively the same as b for the residence's ground floor.

Other principal observations:

- No doors displayed damage.
- 60% of front doors had a postal flap, usually with b ≈ 1 m. Few other doors had a postal flap.
- Small glass units (H and W ≤ 0.25 m) were common
- One door had a cat flap.

6.2.13 Windows

1,607 window units were observed. 3% had H or W > 1.0 m and 69% had both H and W ≤ 0.50 m. Materials for the units' frame, transoms, and mullions were:

- PVC: 75%
- timber: 25%

Other principal observations:

- 49% of units did not open. 100% (all but one) of opening units opened by swinging outwards.
- 1% of units were not rectangular.
- 1% of units displayed minor damage, mainly to the frame. The rest had no damage.
- 99% of units were double-glazed.
- 3% were not simple, flat plates but had an additional design in the glass.
- Two houses and one flat column had large window units with b ≤ 0.8 m. Other window units either had one dimension ≤ 0.25 m or b ≥ 0.8 m.

6.2.14 Other Openings

Other than air bricks and one vent with b = 0.3 m, no large openings were observed with b < 2.1 m. Pipes, ducts, and inlets for cables on ground floors were small, with R ≤ 0.1 m^2. Seven larger ones had b ≥ 2.5 m and one large one had b = 2.3 m. Air bricks had H = 0.08 m and W = 0.2 m and were placed every 1-2 m resulting in an average of 3-5 air bricks per wall. The air bricks had either b ≤ 0.3 m or else were at the top of the ground floor, yielding b ≈ 2.4 m.

6.2.15 Preliminary Comments on Hull

Residential properties have recently been constructed on the most vulnerable areas of Hull to storm surge flood actions. The erratic layout of the communities and the widely disparate orientation of the residences implies a large degree of unpredictability in flood modelling from urban design factors.

The small number of bungalows (4% of houses) implies that people do generally have a means of escape to an upper floor if walls do not collapse. Residents in ground floor flats (33% of flats) could seek refuge in someone else's flat if they are given adequate warning, but would still incur high losses and would not be protected from residence destruction.

Evacuation is also a problem. Hessle Foreshore (117 residences) has one road out and Victoria Dock Village (1,395 residences) has two exit roads, both leading to the same main road. The Wellington Street area has various escape routes by vehicle or on foot.

A major storm surge event in Hull has the potential for severe damage from breaking waves. 93 houses and 182 flats are directly exposed to the shoreline, thus any event near or above the height of the sea defences would subject these residences to the immense forces of any breaking waves. 17 houses and 12 flats are protected by a shingle embankment, whereas the others have a concrete wall, mainly sloping, at the Estuary's edge. The best form of defence against wave actions is not clear but shingle can be turned into debris by waves (Lewis, 1999).

6.3 Observations: Comparing Canvey Island and Hull

In addition to the configuration matrices (Tables 6.1.2.1 and 6.2.2.1), the residences of each case study site are characterised in Table 6.3.1.1.

Table 6.3.1.1: Comparing Canvey Island and Hull

Characteristic	Canvey Island	Hull
age	post-1960	post-1980
basements	none	none
design use	residential	residential
ground to ground floor bottom	0.3 m	0.2 m
loft = storey	yes	yes
relative height	flat	different areas identified
wall type	cavity with brickwork outer leaf and blockwork inner leaf	cavity with brickwork outer leaf and blockwork inner leaf
wall height	2.4 m	2.4 m
wall, door, and window damage	none	none
door and window characteristics	similar	
important other openings	air bricks	air bricks

Π	mean	10th percentile	50th percentile (median)	90th percentile
Canvey Island	28.7 m ± 12.3 m	11.8 m	26.5 m	42.9 m
Hull	23.8 m ± 9.4 m	15.0 m	21.0 m	35.2 m
Canvey Island and Hull	25.8 m ± 10.9 m	13.0 m	23.4 m	38.9 m

A	mean	10th percentile	50th percentile (median)	90th percentile
Canvey Island	61.7 m^2 ± 23.3 m^2	42.0 m^2	57.2 m^2	88.2 m^2
Hull	51.4 m^2 ± 18.6 m^2	32.7 m^2	44.7 m^2	74.9 m^2
Canvey Island and Hull	55.7 m^2 ± 21.2 m^2	34.2 m^2	52.8 m^2	79.1 m^2

These observations illustrate that the residences in both case study sites are similar in terms of their components, A, and Π. The configuration matrices are different indicating that the residence populations are different even though each residence is built in a similar fashion from similar components.

6.4 Revision of Residence Classes

6.4.1 Redefinition of Configuration

One aspect which emerges from Tables 6.1.7.1, 6.1.8.1, 6.2.7.1, and 6.2.8.1 is that neither exterior wall configuration nor the number of storeys or levels correlates well with either A or Π. Some general, expected trends are noted, but strong patterns yielding a solid basis for defining residences are not apparent. Additionally, many exceptions exist to the general trends. Two examples:

- On Canvey Island, two-storey houses with four exterior walls may have small A (55 Hope Road), medium A (17 Sanders Road), or large A (8 Hardy's Way).
- In Hull, three-storey flats with two opposite walls may have small Π (55-57 Lancelot Court), medium Π (37-39 Lancelot Court), or large Π (20-22 Kingston Wharf).

Such diversity in A and Π for a given wall configuration occurs frequently.

The proposed choice of exterior wall configuration and j to define residences (Section 5.4.1) is inadequate. Alternatively, Chapters 8, 9, and 10 indicate that A, Π, and j better define the first-order, physical vulnerability of residences to the ΔP from f_{diff} and v. Chapter 1 notes that this Chapter and Chapters 8, 9, and 10 were developed in parallel, feeding back into each other. One consequence of this feedback is that, at this stage, the definition of residence with respect to flood vulnerability must shift towards examining A, Π, and j without further consideration of exterior wall configuration. The evolution of defining residence configuration for physical vulnerability to floods is illustrated in Table 6.4.1.1.

Table 6.4.1.1: Evolution of Residence Definition

source	variables	notes
Sections 5.2 and 2.4 (past studies)	age	inadequate (Section 5.3)
	configuration: detached, terraced, flat…	inconsistent (Section 5.3)
Section 5.3 (survey method for this work)	j	Chapter 9
	exterior wall configuration: 1, 2opp, 2adj, 3, or 4	improvement of past studies
Section 6.4.1 (post-survey revision based on data)	A	Section 6.4.1 and Chapters 8, 9, and 10
	j	
	Π	
Section 6.4.2 (based on data)	A-Π combinations	Section 6.4.2
	j	

This analysis of residence characteristics indicates that for first-order physical vulnerability to ΔP from f_{diff} and v, a residence may be considered a combination of its components. Each residence component does not yield first-order vulnerability pertinent to a specific residence class, but first-order vulnerability pertinent to all residences.

This statement does not imply that residences of different configurations have exactly the same vulnerability level. Rather, it states that the specific vulnerabilities of a component—such as a painted timber door, a 1.0 m × 0.5 m window, or a first floor—contribute the same level of vulnerability irrespective of the residence's configuration. A residence with high A and Π has a different level of vulnerability than a residence with low A and Π, but each ground floor window of similar geometry contributes the same vulnerability to each residence.

This approach may be valid for only the vulnerability characteristics explored here. Other vulnerabilities requiring a different set of observations may require a different method. For example, to determine social vulnerability, the meaning of home becomes important (Coates and Fordham, 2000). A person's or family's attachment to their home cannot be determined from the combination of residence components. The key here is that this building survey was conducted with a specific purpose: to identify the features of residential buildings which are most physically vulnerable to ΔP from f_{diff} and v. For this purpose, residence classes defined by A, Π, and j are appropriate and a residence may be viewed as a combination of its components.

The similarities between the two case study sites are also apparent in Table 6.3.1.1, including for A and Π. The reason is most likely due to the relatively young age of the residences in both locations (Sections 6.1.10 and 6.2.10). Therefore, for identifying a typical residence for this study by A and Π, taking the combination of Canvey and Hull is reasonable.

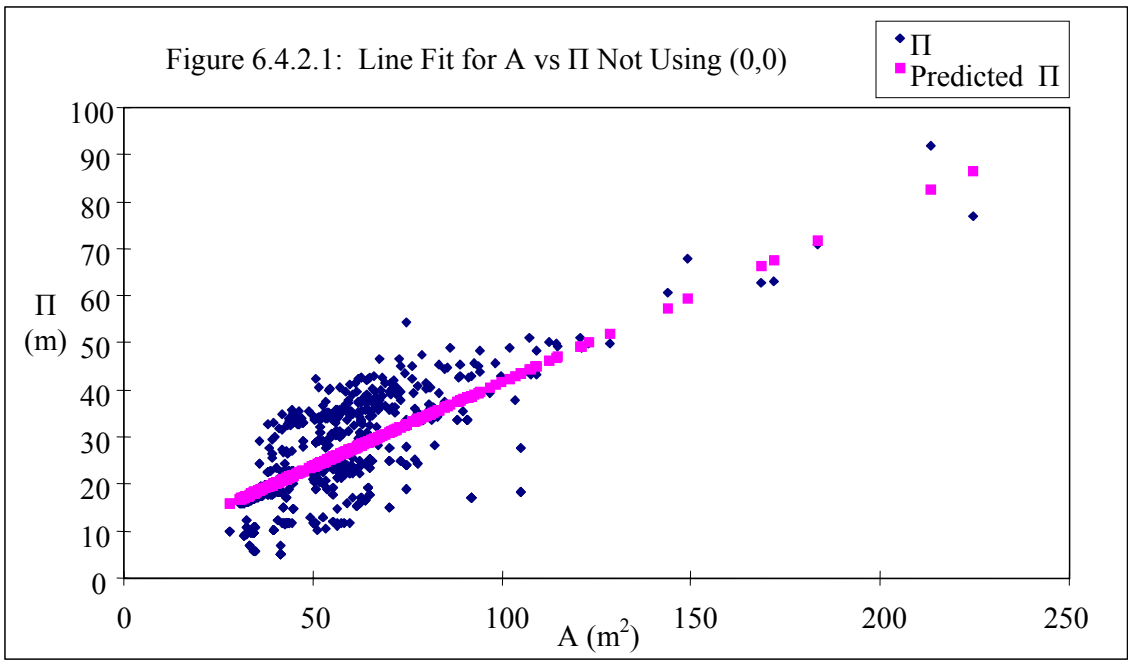
Combining the data sets from both case study sites implies that the work here is effectively for typical residences rather than exclusively for the two case study sites. The results from this study, such as the vulnerability matrices (Chapter 12), may be transferable to other locations, but only if a typical residence is similar to a typical residence in Canvey Island and Hull. If this situation exists, then by determining the local proportion of each A-j-Π class, a relatively straightforward task, the flood vulnerability model may be rapidly applied.

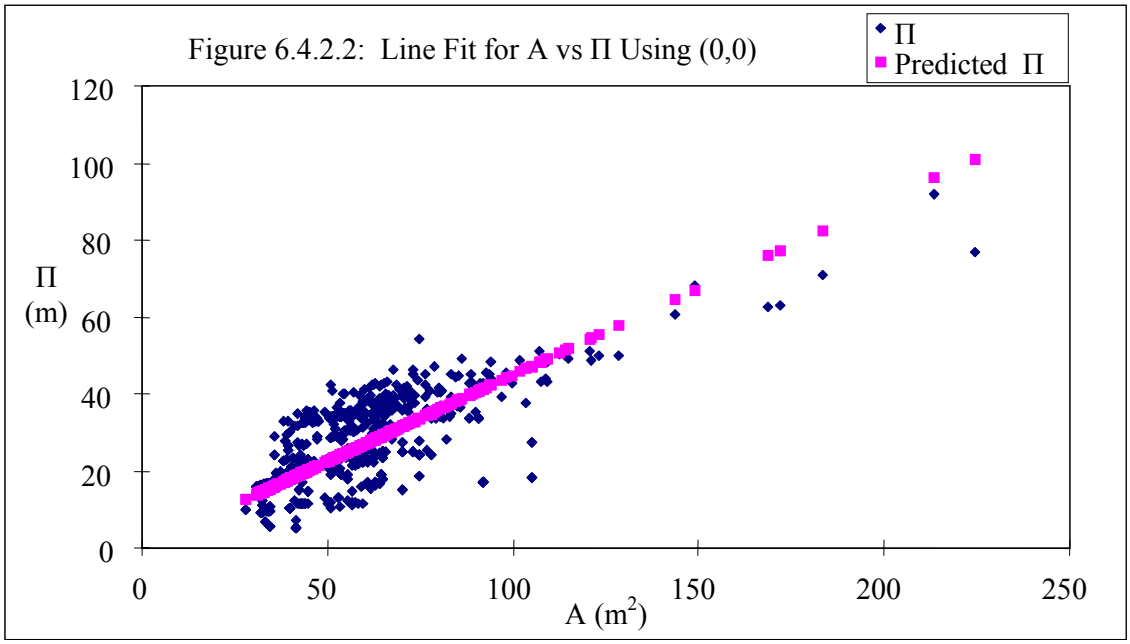
Flood consequences must be analysed separately because flood scenarios, and thus the hazard characteristics experienced in each case study site, would be different. A regional flood vulnerability model is developed for certain residence types in coastal, eastern England, but the output must be tailored to the local situation. For example, the vulnerability matrices (Chapter 12) may be transferable to other locations, but differing relative height of communities would need to be factored in by adjusting f_{diff}.

6.4.2 Correlating A and Π

A loose connection between A and Π would be expected. As A increases, Π could be larger to contain that larger A. Therefore, a correlation test was run on A and Π. Without using the point A = 0 and Π = 0, $r^2 = 0.49$ (Figure 6.4.2.1). Using the point A = 0 and Π = 0, $r^2 = 0.45$ (Figure 6.4.2.2).

Regression r^2 from both graphs indicates a reasonably strong correlation, but large clusters of residences appear above and below the predicted line in the vicinity of A = 50 m^2. These residences exist; they are not spurious measurements. The explanation is straightforward: a residence with one or two exterior walls may have A as large as desired without changing Π much. Any observed correlation must have a good explanation of causation. In this case, an explanation for non-causation exists supported by the reality of definite observations. Thus, A and Π cannot be directly correlated in order to combine them into a single variable for





defining residence classes. Instead, they must be viewed as a linked pair with some potentially infeasible combinations.

To represent all residences, typical parameter values are required along with values which are small and values which are large. A typical value for a parameter is given by the median or 50th percentile (Ross, 2000; Walpole *et al.*, 1998). For small and large values, the 10th and 90th percentiles are chosen respectively. Thus, values for small, typical, and large A may be combined with values for small, typical, and large Π to produce nine possible A-Π combinations for each j in order to define residence class.

From the A and Π statistics in Table 6.3.1.1, a compromise between Canvey Island and Hull is sought to represent residences in coastal, eastern England. The following factors must be considered:

- Canvey has approximately ten times as many vulnerable residences as Hull.
- The Canvey photographic survey covered 64% the amount of residences as the Hull photographic survey.
- The error in Π is less than ± 2 m and the error in A is on the order of 10% (Section 5.4.2).
- Differences between flats and houses are negligible in comparison to the errors of the measurements.

Therefore, the values chosen to represent residences in coastal, eastern England are in Table 6.4.2.1.

Table 6.4.2.1: A and Π Representing Coastal, Eastern England

	10th percentile (small)	50th percentile (median, typical)	90th percentile (large)
Π (m)	12	25	41
A (m^2)	38	55	84

Examining the feasibility of each combination indicates that eight of the nine A-Π pairs are realistic and exist (Table 6.4.2.2).

Table 6.4.2.2: Realistic A-Π Pairs for Coastal, Eastern England

Π (m)	A (m^2)	Π/A (m^{-1})	Sample Configurations (exterior walls; lengths in m)	Example
12	38	0.32	w2adj 5×7 w2opp 6×6	Hull, 2 Riverside Court
12	55	0.22	w1 12×5 w2opp 6×10	Canvey, 310 Furtherwick Road
12	84	0.14	w1 12×7 w2opp 6×14	Canvey, Cherry Court
25	38	0.66	w3 10×4 w4 8×5	Canvey, 40 Berg Avenue
25	55	0.45	w3 9×7 w4 10×5	Canvey, 6 Vicarage Close
25	84	0.30	w2opp 12×7 w3 9×9	Hull, 4 to 6 Lancelot Court
41	38	1.08	w1 40×1 w2opp 19×2	None
41	55	0.75	w4 7×10 with a garage	Canvey, 4 Corona Road
41	84	0.50	w4 12×7	Hull, 67 Harbour Way

For classifying each residence according to a configuration in Table 6.4.2.2, the ratio Π/A was also considered. This ratio assists in identifying structure morphology (Steadman *et al.*, 1991). Steadman *et al.* (1991) use exterior wall area rather than Π for the ratio, but since this study uses H = 2.4 m for all residences, Π/A as an indicative value is valid. Π/A was particularly helpful in resolving cases where the choice of configuration through only A and Π was ambiguous.

Since the eight A-Π combinations for each j define residences with respect to first-order physical vulnerability to ΔP from f_{diff} and v, the A-j-Π configurations represent flood vulnerability classes. Results from classifying each residence in the photographic survey appear in the following five tables. Table 6.4.2.7, a summary interpretation of the other four tables, matches up wall configuration with an A-Π pair in order to generalise the meaning of each A-Π pair.

Table 6.4.2.3: Distribution of Vulnerability Classes on Canvey Island

Π (m)	A (m^2)	1 storey	2 storeys	3 storeys	Total
12	84	0 (0.0%)	4 (1.2%)	3 (0.9%)	7 (2.1%)
12	55	6 (1.8%)	32 (9.6%)	2 (0.6%)	34 (10%)
25	84	1 (0.3%)	14 (4.2%)	2 (0.6%)	18 (5.4%)
12	38	1 (0.3%)	8 (2.4%)	2 (0.6%)	34 (10%)
25	55	30 (9.0%)	98 (29%)	6 (1.8%)	82 (24%)
41	84	23 (6.9%)	40 (12%)	5 (1.5%)	76 (23%)
25	38	11 (3.3%)	16 (4.8%)	0 (0.0%)	63 (19%)
41	55	14 (4.2%)	17 (5.1%)	0 (0.0%)	21 (6.3%)
Total		86 (26%)	229 (68%)	20 (6.0%)	335 (100%)

Table 6.4.2.4: Exterior Walls by Vulnerability Class on Canvey Island

Π (m)	A (m^2)	1 storey	2 storeys	3 storeys
12	84	--	4 are w1	1 is w1 2 are w2opp
12	55	6 are w2opp	2 are w1 30 are w2opp	1 is w1 1 is w2adj
25	84	1 is w3	1 is w2opp 9 are w2adj 4 are w3	2 are w3
12	38	1 is w4	6 are w2adj 1 is w3 1 is w4	1 is w1 1 is w2adj
25	55	19 are w3 11 are w4	14 are w2adj 1 is w2opp 63 are w3 21 are w4	1 is w1 1 is w2adj 2 are w3 2 are w4
41	84	2 are w3 21 are w4	1 is w3 39 are w4	5 are w4
25	38	11 are w4	8 are w3 8 are w4	--
41	55	12 are w4 1 is w3	17 are w4	--

Table 6.4.2.5: Distribution of Vulnerability Classes in Hull

Π (m)	A (m^2)	1 storey	2 storeys	3 storeys	4 storeys	Total
12	84	0 (0.0%)	11 (2.2%)	0 (0.0%)	0 (0.0%)	11 (2.3%)
12	55	5 (1.0%)	4 (0.8%)	3 (0.6%)	0 (0.0%)	12 (2.5%)
25	84	0 (0.0%)	3 (0.6%)	27 (5.6%)	8 (1.7%)	38 (7.9%)
12	38	0 (0.0%)	150 (31%)	21 (4.3%)	0 (0.0%)	171 (35%)
25	55	19 (3.9%)	78 (16%)	20 (4.1%)	0 (0.0%)	117 (24%)
41	84	0 (0.0%)	9 (1.9%)	0 (0.0%)	0 (0.0%)	9 (1.9%)
25	38	0 (0.0%)	49 (10%)	0 (0.0%)	0 (0.0%)	49 (10%)
41	55	0 (0.0%)	72 (15%)	4 (0.8%)	0 (0.0%)	76 (16%)
Total		24 (5.0%)	376 (78%)	75 (16%)	8 (1.7%)	483 (100%)

Table 6.4.2.6: Exterior Walls by Vulnerability Class in Hull

Π (m)	A (m^2)	1 storey	2 storeys	3 storeys	4 storeys
12	84	--	10 are w1 1 is w2adj	--	--
12	55	5 are w1	4 are w2opp	3 are w2opp	--
25	84	--	3 are w4	9 are w1 1 is w2adj 8 are w2opp 9 are w3	2 are w1 2 are w2opp 4 are w3
12	38	--	3 are w2adj 28 are w2opp 119 are w3	8 are w2opp 13 are w3	--
25	55	18 are w3 1 is w4	1 is w2adj 8 are w2opp 12 are w3 57 are w4	6 are w2opp 14 are w3	--
41	84	--	2 are w3 7 are w6	--	--
25	38	--	49 are w3	--	--
41	55	--	72 are w4	4 are w3	--

Table 6.4.2.7: Interpretation of A-Π Combinations

Π (m)	A (m^2)	Configuration Description
12	84	large residence with 1 or 2 exterior walls
12	55	medium residence with 1 or 2 exterior walls
25	84	large residence with 2, 3 or 4 exterior walls
12	38	small residence with 1, 2, 3, or 4 exterior walls
25	55	medium residence with 2, 3, or 4 exterior walls
41	84	large residence with 3 or 4 exterior walls
25	38	small residence with 3 or 4 exterior walls
41	55	medium residence with 3 or 4 exterior walls

6.5 Further Discussion

This survey focused on first-order vulnerability of residences to flood actions of ΔP from f_{diff} and v. Residences may be designed with many other considerations, including aesthetics, durability, cost, energy use, energy supply vulnerability, water use, water supply vulnerability, ventilation, air circulation, thermal characteristics, non-flood perils (natural, such as lightning and earthquakes, and non-natural, such as theft or gas explosions), acoustic properties, sound penetration, internal lighting, light penetration, olfactory properties, textures, and all these characteristics' gradients over space and through time. Although recommendations are made here for reducing residence vulnerability to specific flood actions, other concerns could take precedence when making design choices. In permitting other concerns to take precedence over flood vulnerability, understanding the effect on flood vulnerability is important to ensure that flood vulnerability is not exacerbated through ignorance.

Figure 6.5.1:
House on Southern Canvey Island

For example, the house in Figure 6.5.1 on southern Canvey Island is the first property in its row behind the sea wall and was visited on 30 November 1999. The ground floor was raised just over 1 m from ground level and the ground floor windows were placed high up and were small. A possible explanation for these design choices is to increase flood resistance. The occupant of the house across the street stated on 30 November 1999 that the design was chosen to enable a view of the Thames Estuary over the sea wall. He stated that bedrooms were on the ground floor in order to make the view available from daytime rooms such as the lounge and dining area. The aesthetic imperative has apparently inadvertently reduced physical flood vulnerability by raising the structure but has inadvertently increased human flood vulnerability by placing the sleeping area in the most flood-vulnerable part.

Additionally, the residence survey indicated that the data collected can change surprisingly quickly, at the time scale of weeks or months, due to occupant modifications, the construction of new developments, and the shifting use of some buildings (e.g. from commercial to

residential). Differences in addresses between the Royal Mail (2001-2) and Digimap (2001-2) illustrated inconsistencies amongst data sources. Such data sources also occasionally differed from photographs, even to the extent of exterior wall configurations being different. Additionally, several dozen residences appeared in neither Royal Mail (2001-2) nor Digimap (2001-2) during this study, yet exist. They seem to have been built in the months immediately prior to the survey, indicating how quickly the data change.

Following a major disaster such as a flood or windstorm, damage, repairs, and reconstruction may substantially alter the residence features, further reducing the accuracy of the data presented here. Although the data are not robust in time, they are robust in space for parts of coastal, eastern England as seen by comparing Canvey Island and Hull (Section 6.3).

Other potential inaccuracies were extreme values which were not recorded in the photographic survey because they were not encountered. Window units up to a size of 2 m $\times$ 4 m were observed on Canvey Island, but not in any residence in the photographic survey. On the other hand, some extreme values were observed such as the ground to ground floor distance of 0.6-0.7 m observed for Riverside Court and 20 St. Mary's Court in Hull. Interpreting unusual observations must proceed with caution in attributing them to a specific vulnerability or design issue, as noted earlier with the house on southern Canvey Island.

6.6 Conclusions

The results in this Chapter illustrate the application of the method developed for surveying residences to determine their first-order physical vulnerability to ΔP from f_{diff} and v. The definition of residence has been revised to eliminate configurational classes in favour of the numerical properties of A, j, and Π. Houses and flats have been combined to yield 32 housing types based on eight representative A-Π pairs and $j \in \{1, 2, 3, 4\}$.

The two case study sites comprise mainly modern residences. This result simplifies the vulnerability analyses. For example, only cavity walls and double-glazed units need to be considered. Additionally, it is unlikely that many of the residences surveyed have experienced floods, so immediate validation is challenging but the analyses are important for estimating potential, future flood consequences. The focus on modern residences, however, yields results limited in their applicability to the observed residence types.

7. Flood Failure Flowchart Prelude

7.1 Introduction

Chapter 6 established the types, numbers, and materials of residence components in the case study sites. Chapter 3 established the flood actions on these residence components to be examined. This Chapter examines the consequences of the flood actions in order to determine how failure in residence components, and thus the entire residence, may occur. The aim of this Chapter is to predict the components which would be most likely to fail under given f_{diff}-v scenarios and to produce a flowchart indicating likely failure pathways.

Tests in British Standards will be frequently outlined in developing this flood failure flowchart. Many British Standards referenced may not be applicable because:

- The flood actions examined in detail in this study are not always considered in detail by the British Standards.
- The assumption must be made that residences and residence components comply with the British Standards.
- The assumption must be made that little change has occurred in the British Standards through time.

These concerns are fully addressed in Section 7.5 and Chapter 11.

Only residence components observed in the case study sites are examined: windows (glass and non-glass components), walls, doors (glass and non-glass components), floors, foundations, and roofs. Basements, for example, are not considered because none were observed. To begin, the rate of flood water infiltration inside a residence is examined to estimate the pressure differentials which could result from water rising outside a residence.

7.2 Flood Infiltration Rate

Various scenarios for all rates of water rise outside the residence, from negligible to instantaneous, can be envisioned. The water will start infiltrating into the residence as soon as it contacts the outside. As the flood water rises inside, it balances ΔP from external f_{diff}, thereby reducing direct hydrostatic pressure. Establishing the rate of rise of flood water inside the residence in comparison to the rate of rise of flood water outside the residence helps to determine the magnitude and duration of ΔP imposed on the residence. Further investigation

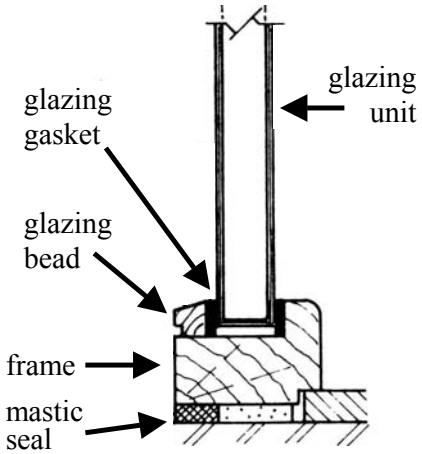
of hydrostatic ΔP, and justification of any resultant damage, would be dependent on the flood rise rate inside the residence being sufficiently slow to permit ΔP to cause damage to the residence. Flood rise rate is explored in Chapter 8.

7.3 Window Failure

7.3.1 Introduction

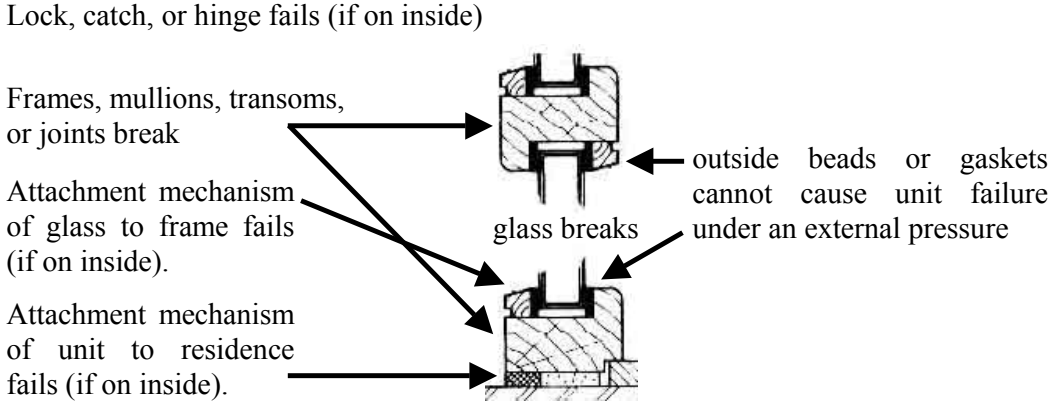
Figure 7.3.1.1 shows a typical window, but many variations exist. Window units under an external ΔP could experience failure (Figure 7.3.1.2) due to glass; the lock, catch, or hinge; the mullions, transoms, frame, or joints; the mechanism for attaching the frame to the wall (e.g. mastic seal or screws); or the mechanism for attaching the glass to the frame (e.g. gasket or bead). Simultaneous failure is unlikely, as normally the weakest component would fail before the others. Thus, this Section aims to establish the weakest component or components.

Figure 7.3.1.1: A Typical U.K. Residential Window (modified from Chudley, 1995)



Interactions and secondary failures are feasible. For example, a frame may bend under ΔP, imposing stress levels in the glass higher than would be predicted by examining each component individually. A mullion or transom which fractures could crack or scratch glass, weakening the material. Warping frames, mullions, or

Figure 7.3.1.2: Potential Failure Modes of Window Units (Based on Figure 7.3.1.1, but the window is obviously unrealistic)



transoms may also impose stress levels on screws, beads, gaskets, or seals. These interactions and secondary failures are discussed in further detail where feasible.

7.3.2 Glass

As detailed in Chapter 10, a literature review indicated that failure ΔP for glass units observed in the case study sites is generally between 1 kPa and 10 kPa. This range warrants further exploration of glass failure, examined in Chapter 10. Chapter 10 notes that many units found in residences may withstand pressures much higher than 10 kPa, but these units are shown to be unimportant for developing the vulnerability matrices described in Chapter 3.

7.3.3 Locks, Catches, and Hinges

In the case study sites, approximately half of all units did not open and over 98% of opening units opened by swinging outwards (Chapter 6). The locks, catches, or hinges on such window units cannot break under an external ΔP.

The relevant British Standard is BS 3621 : 1998 which tests window locks using forces of 9 kN to 13.5 kN. In contrast, BS 6375-2 : 1987 and BS 7950 : 1997 use 0.2 kN to 3.0 kN, applied without shock, for accidental and mechanical load tests on other window components. These forces are approximately one order of magnitude lower than those in BS 3621 : 1998 indicating that locks and catches are relatively strong compared to other window components. The forces in these British Standards are applied for a specific duration on the order of a few minutes. The applicability of BS 7950 : 1997 is discussed further in Section 7.3.5.

One scenario to consider is if the lock or catch were not fastened properly. Low ΔP could force open the few inward-opening units while buoyancy actions combined with localised currents might open an outward-opening window. This situation is analogous to windows being left open, not just unlocked, thereby permitting flood water to enter without any window component failing. Estimating the numbers of open or unlocked windows for flood scenarios cannot be done with sufficient accuracy because important criteria include season, time of day, burglary rate in the community, burglary experience of the occupant, and local security culture. The assumption made for this study is that all window units would be shut and locked when the flood strikes. Depending on the results of the analysis, flood

management recommendations could be formulated regarding the most appropriate action for occupants to take with respect to closing and locking windows.

Another scenario to consider would be following a slow-rise flood, if flood water outside an inundated residence were to drain rapidly. If water exits a residence slowly, then the residence could remain filled with water with no equalising pressure outside. The outward opening window units may then become vulnerable to locks, catches, or hinges breaking. Such a case would be rare and damage would likely be significant already. For the first-order calculations sought here, this case is not considered further.

7.3.4 Mullions, Transoms, Frames, and Joints

PVC and timber were the most common materials used for window mullions, transoms, and frames in the case study sites, representing more than 96% of all units (Chapter 6). BS 6375-2 : 1987 and BS 7950 : 1997 test windows for accidental and mechanical loads using forces of 0.2 kN to 3.0 kN applied without shock to one section at a time. The forces in these British Standards are applied for a specific duration on the order of a few minutes. The applicability of BS 7950 : 1997 is discussed further in Section 7.3.5.

7.3.5 Attachment Mechanisms

Methods for attaching the window unit frame to the rest of the residence would be screws into the brickwork, brackets into the mortar, or sealing adhesives. Rare cases have been observed where the connection between the unit and the residence failed. In referring to gas explosions, Alexander and Taylor (1973) write "where relatively strong panes occur it is possible for the whole window to be ejected before the glass panes fracture, but this depends on the type of fixing". Blyth (1989) notes that most joining components such as screws, nails, and bolts can withstand an approximate load of 10 kN.

The mechanism for attaching the glass to the frame could also fail. Timber frames tend to use a sealant bead or putty on either the inside or outside (McEvoy, 1994) whereas PVC units "always employ gaskets to retain the glazing" (McEvoy, 1994). For PVC units, the tests in Section 7.3.4 also apply to a gasket failing. For timber units, a bead or putty on the outside failing under an external ΔP would not permit water to enter directly.

A bead or putty on the inside failing under an external ΔP would permit the entire glass pane to be pushed inside the residence. Many, but not all, timber-framed units also have some form of gasket or support between the bead or putty and the glass (see McEvoy, 1994), so the tests in Section 7.3.4 would apply. Baird (2002) notes that putting the bead on the inside of a timber unit is not recommended and, as with putty on the inside, occurs rarely.

BS 7950 : 1997 uses 1.0 kN to 3.0 kN for accidental and mechanical load tests for an entire unit but refers to "enhanced security" which is not specifically defined. Although "compliance with this standard cannot be claimed in respect of individual component parts" (BS 7950 : 1997), the expectation that the load tests would be passed by the entire in-place unit is indicative of the loads which each component could withstand. One concern is that "BS 7950 is based on the premise that if a burglar fails to gain access ONCE and within 3 minutes, he or she will go elsewhere" (Baird, 2002) which does not apply to floods.

7.3.6 Conclusions

The information available on the strength of the non-glass components of windows, particularly the British Standards, does not provide a basis for a detailed comparison of component strengths. Expert judgement (Baird, 2002; Chisholm, 2001; Keiller, 2001; McEvoy, 2001-2) suggested that failure in non-glass components would be unlikely for the sizes of units in residential properties. In analysing damage from gas explosions, Alexander and Taylor (1973) note that glass breaking tends to be the damage most frequently sustained by windows, although the pressures imposed by explosions are different from those imposed by floods. Section 7.4.2 discusses the British Standards relevant to explosion resistance, BS EN 13123-1 : 2001 and BS EN 13124-1 : 2001.

This small amount of evidence suggests that further analysis of potential failure in windows should focus on glass. Glass failure will be examined in detail in Chapter 10, but failure in non-glass components has not been completely ruled out yet.

7.4 Non-Window Failure

7.4.1 Wall Failure

As detailed in Chapter 9, a literature review indicated that failure ΔP for wall panels is generally between 1 kPa and 10 kPa. This range warrants further exploration of wall failure, examined in Chapter 9.

7.4.2 Doors

Chapter 6 provides relevant data on doors. In the residences in the case study sites, 71% of doors opened by swinging inwards while 22% opened by sliding sideways. 86% of doors had glass units which are treated similarly to glass units in windows (Section 7.3 and Chapter 10). Otherwise, PVC and timber were the most common materials for doors and for frames around door glass units. b (the height above ground) for ground floor doors was equivalent to that of walls.

BS EN 950 : 1999 and PAS 23-1 : 1999 describe impact tests for doors, excluding glass parts. The tests use a concentrated load and the impact is measured in energy terms. The test applies to the entire door assembly which covers the door material, hinges, and locks.

BS EN 13123-1 : 2001 and BS EN 13124-1 : 2001 discuss the classification of doors, windows, and shutters to explosion resistance, simultaneously covering all components of the door, window, or shutter under consideration. The four categories for classifying explosion pressure resistance are peak pressures of 50 kPa, 100 kPa, 150 kPa, and 200 kPa. These categories are not absolute requirements, but requirements for claiming a specific explosion pressure resistance rating for a door, window, or shutter.

These standards do not greatly assist in determining the strength of doors, windows, and shutters in withstanding ΔP from f_{diff} and v. First, an explosion's peak pressure lasts milliseconds, at least three orders of magnitude less than the duration of peak pressures due to hydrostatic and hydrodynamic ΔP. Second, the door, window, or shutter may be accorded a rating even if certain failures occur at the given peak pressure.

BS 5872 : 1980 describes various mechanical tests for different door locks and latches. The forces, varying from approximately 0.5 kN to approximately 4.0 kN, are applied along different axes on different parts of the locks and latches.

As with windows, possible scenarios include doors left open or unlocked. Estimating the numbers of open or unlocked doors for flood scenarios cannot be done with sufficient accuracy for similar reasons to windows (Section 7.3.3). The assumption made for this study is that all doors would be shut and locked when the flood strikes. Depending on the results of the analysis, flood management recommendations could be formulated regarding the most appropriate action for occupants to take with respect to closing and locking doors.

Also similarly to windows, following a slow-rise flood, flood water outside an inundated residence may drain rapidly leaving the residence filled with water with no equalising pressure outside. In this case, vulnerability would not increase due to the small number of doors which open outwards. Irrespective, as with windows, for the first-order calculations sought here, this case is not considered further.

7.4.3 Floors

Bramley and Bowker (2002) note that "in the U.K., common floors are either of solid (concrete) or suspended (timber or concrete) construction". DTLR (2002) lists only these three floor types as well. These alternatives are considered in the flood infiltration rate calculations (Chapter 8). Figures 7.4.3.1 and 7.4.3.2 display typical examples of solid and suspended floors respectively.

Water pressure from below could potentially break both timber and concrete floors. The pressure could be imparted by groundwater rise or, in the case of suspended floors, by water infiltrating through air bricks and filling the space below the floor. A case of flood water apparently cracking concrete in order to escape was witnessed (Figure 7.4.3.3), although the situation in a residence would be different. In Figure 7.4.3.3, the pressure underneath the pavement was substantial even for flood water. The pressure arose from a flow similar to that observed in Figure 9.1.2.3. and no other escape route for the water was apparent. For residences, similar pressures might conceivably be experienced, but other potential escape routes would be expected, such as by breaking other residence components. For groundwater rise, suspended floors also have air bricks as escape pathways.

Figure 7.4.3.1: A Typical U.K. Residential Solid Concrete Floor (modified from Chudley, 1995)

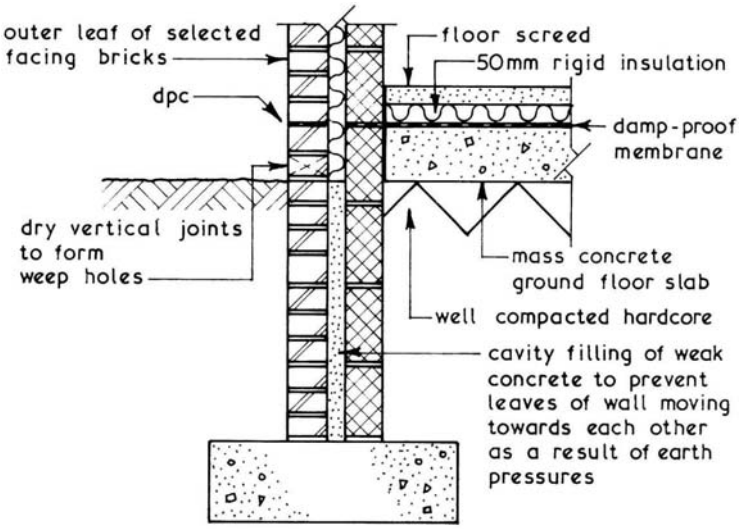
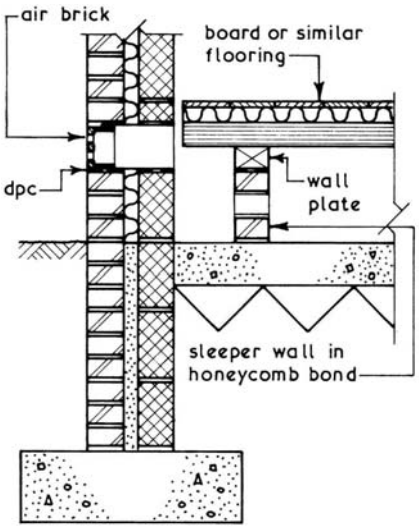


Figure 7.4.3.2: A Typical U.K. Residential Suspended Timber Floor (modified from Chudley, 1995)



DTLR (2002) relates one anecdote of a house in England which suffered a different cracked concrete floor slab in two different floods. Nevertheless, DTLR (2002) comments "modern solid concrete floors with damp proof membranes are generally regarded as the most flood resistant floor type…concrete floor slabs are unlikely to be significantly damaged by flooding". EA/CIRIA (2001b) states "solid flooring is much more resistant to flood damage than floor boards". In both documents, it is unclear which flood actions have been considered. DTLR (2002), however, later states "concrete floors, provided they are properly designed to withstand the pressure of flood water, can provide an effective seal to prevent water seeping up from the ground". Properly designed implies "leaving a small gap around the [floor's] edges to relieve the pressure of the water" (DTLR, 2002).

Figure 7.4.3.3: Water Cracking a Pavement to Escape, Malton, U.K., November 2000

The little evidence available suggests that floors breaking frequently due to ΔP from f_{diff} and v is unlikely. Buoyancy is much more likely to cause failure, such as by causing timber floor joists or surface boarding to float.

Nonetheless, floor failure due to strictly ΔP from f_{diff} and v—or even from groundwater rise—is feasible and the floor may need to be replaced. At DS2 (Table 1.4.1), however, most timber floors would need to be replaced, so little difference in damage occurs whether or not the floor breaks. At DS2, concrete floors would require months to dry properly, may crack during the drying process, and may need to be replaced anyway, particularly parts such as insulation and screed (DTLR, 2002; EA/CIRIA, 2001a; BRESL, 1996; BRE, 1997). Again, little difference in damage occurs whether or not the floor breaks.

Flood actions may affect floors, but would appear to be unlikely to significantly affect the resulting damage or repair cost. Although uncertainties are present, the best available evidence suggests that floors breaking is unlikely to be significant for residence failure at the first order due to ΔP from f_{diff} and v. One major difference between the floor breaking or not breaking would be the resultant debris which could cause further damage.

7.4.4 Foundations

The foundation "is defined as that part of the structure in direct contact with the ground and which transmits the load of the structure to the ground" (Tomlinson, 1995; see also Figures

7.4.3.1 and 7.4.3.2 which display a typical foundation, although others exist as shown in Chudley, 1995). Any instability or destruction of the foundation has severe implications for the integrity of the entire structure. Even if the siting is shallow, most of the foundation normally sits below ground level and this situation was observed for most residences in both case study sites. ΔP from f_{diff} and v would rarely impact the foundation directly.

Foundations tend to be highly vulnerable to flood erosion actions (Section 3.2.4). Soil may be washed away destabilising the foundation. Buoyancy actions are a concern too. Buoyancy problems with foundations arise even without flooding. Foundations are usually designed so that the weight of the structure on top is greater than the expected buoyancy force, thereby keeping the structure in place (Tomlinson, 1995). Rising ground water or increased soil saturation from flooding may produce pressures which move or tilt the structure. Significant damage to the structure may result and a large cost may be incurred to maintain stability.

Foundation damage from erosion or buoyancy actions may dominate observed damage except where structural collapse has occurred due to ΔP from f_{diff} and v. Pressures on or movement of foundations may also weaken other parts of the residence resulting in increased f_{diff}-v damage. As noted in Chapter 3, erosion and buoyancy actions are not detailed in this study. Therefore, damage to or exacerbated by foundations is not considered in detail in this study.

7.4.5 Roofs

BS 6399-3 : 1988 is the relevant British Standard for imposed roof loads and suggests designing for minimum imposed uniform loads of 0.6 kPa to 1.5 kPa, depending on the roof type. Roofs are weak compared to other residence components, particularly glass and walls. Nevertheless, if flood water is infiltrating, or if waves or debris are penetrating, through a residence roof, then the residence would be expected to be subjected to relatively significant flood actions which would be likely to cause failure in other residence components. The equivalent of complete destruction (DS5) would be likely, although is not certain. This assumption is revisited in Chapter 11.

7.4.6 Summary Thus Far

The discussion in Section 7.4 has identified wall failure and glass failure as the principal failure modes for a first-order analysis of the effects of ΔP from f_{diff} and v on residences in the case study sites. Failure of roofs and of non-glass components in doors and windows has not been completely ruled out. Neither floor nor foundation failure is considered significant enough to require further analysis.

These modes were analysed based on the need to examine ΔP from f_{diff} and v in detail amongst all possible flood actions. The flood rise rate is important too, because it indicates the speed with which ΔP from hydrostatic actions would reach zero as flood water enters the residence and balances hydrostatic ΔP outside the residence.

7.5 Conclusions

This Chapter frequently references tests outlined in British Standards. Interpreting these tests is not straightforward. Many of the test loads are concentrated forces, sometimes applied with shock and sometimes applied without shock. In contrast, the flood actions examined in detail here are pressures distributed over a relatively large area, potentially for a relatively long duration. Debris may produce concentrated forces analogous to the forces suggested in some standards.

Furthermore, the main implicit assumptions in this analysis are that the British Standards are complied with and that little change has occurred in the British Standards through time. The former assumption is challenging to verify. The latter assumption is certainly true in some cases, but is uncertain in others. For example, BS 5544 : 1978 and BS 5872 : 1980 were statutory when the majority of residences surveyed were being constructed, although BS 5544 : 1978 is a specification for a specific type of glazing rather than a requirement for all glazing. Similarly, Chapter 8 references the contemporary BS EN 1027 : 2000 but BS 5368-2 (EN 86) : 1980, the predecessor to BS EN 1027 : 2000, is still in force and little changes between the two documents. In contrast, some newer standards such as BS EN 950 : 1999 do not have similar standards which would have applied previously.

Irrespective of these inconsistencies, comparing the standards provides a degree of measure of the expected relative strength amongst residence components. While all details in the

standards should not be taken as completely indicative of the residences surveyed, the standards are the most reliable and comprehensive source available for the information sought here. Coupled with expert judgement, as done here and in subsequent Chapters, this information must be interpreted as best as possible.

This interpretation is summarised in Table 7.5.1. Certain failure modes must be examined further: flood rise rate (Chapter 8), wall failure (Chapter 9), and glass failure (Chapter 10). Failure of floors and foundations does not require further analysis. Failure of roofs and of non-glass components in doors and windows appears to be unlikely, but shall be revisited in Chapter 11 after the more detailed analyses. Therefore, at this stage, enough information to produce a formal flood failure flowchart is not available. The presentation of the flood failure flowchart is deferred until Chapter 11.

Table 7.5.1: Summary of Initial Analysis of Failure Modes

Failure Mode	Detailed Analysis Required?
Flood infiltration.	Yes. See Chapter 8.
Window glass.	Yes. See Chapter 10.
Window locks/catches or hinges.	Unclear. Clarified in Chapter 11.
Window mullions, transoms, frames, or joints.	Unlikely. Clarified in Chapter 11.
Window attachment mechanisms.	Unlikely. Clarified in Chapter 11.
Walls.	Yes. See Chapter 9.
Doors.	Unclear. Clarified in Chapter 11.
Floors.	No.
Foundations.	No.
Roofs.	Unclear. Clarified in Chapter 11.

8. Flood Rise Rate Inside a Residence

8.1 Theoretical Justification

8.1.1 Introduction

Flood scenarios for all possible rates of water rise outside a residence, from negligible to instantaneous, can be envisioned. ΔP from f_{diff} and v would force water into the residence through any cracks or openings. The water would then rise inside the residence reducing f_{diff} and consequently ΔP. Knowing the rate at which water infiltrates from outside to inside a residence under different scenarios as well as the rate of water rise inside a residence is thus necessary for determining the range of possible ΔP which could be imposed on residences. The flood infiltration rate is termed FIR in m^3/s and the flood rise rate inside a residence is termed FRR in m/s.

Past studies on the rate of water infiltration into buildings were not found. The potentially relevant British Standards are BS 6375-1 : 1989, BS EN 1027 : 2000, and BS EN 12865 : 2001. In discussing weather-tightness of windows and other building components, water infiltration rates are not considered, so these standards do not assist in calculating FIR or FRR.

Instead, infiltration studies of a different fluid, air, are adapted for applicability to water infiltration. These studies emerge mainly from the literature on natural ventilation and air leakage rates. They focus on air flow under external wind pressure through small openings termed cracks. Examples of cracks are the space between a window frame and the wall, the gap underneath a door, and the porosity of brickwork and render.

Fine cracks of this type do not yield fully developed flow (Walker *et al.*, 1997); yield flow which is transitional between laminar and turbulent (Orme *et al.*, 1998); and do not provide ideal flow conditions due to surface roughness. Two principal methods are available for calculating crack flow rates of air into residences by accounting for these non-ideal factors: the power law and the quadratic formula.

8.1.2 Past Studies: Power Law

The general power law is $Q_{crack} = C\Delta P^n$ for which:

- $C = C_{drag}R(2/\rho)^n$ where C_{drag} indicates surface roughness and ρ refers to any Newtonian fluid, including water and air.
- $\frac{1}{2} < n < 1$ where $n = \frac{1}{2}$ represents fully developed turbulent flow and $n = 1$ represents fully developed laminar flow (Orme *et al.*, 1998; Walker *et al.*, 1997). For crack flow, n is generally taken as 0.6 or 0.66, but for large openings such as postal flaps, $n = \frac{1}{2}$ would be used (Li *et al.*, 2000; Liddament, 1996).

For air flow through residence cracks, Orme *et al.* (1998) provide data for values of C and n compiled from empirical sources. Orme *et al.* (1998) is an updated version of the main data source used for CIBSE (1999), the standard U.K. guide for infiltration calculations.

One input towards determining whether or not air flow through a crack and water flow through a crack are similar would be to compare Reynolds number (Re). This step is only an approximation because Re calculations apply to fully developed flow and use $n = \frac{1}{2}$. For air and water in Re calculations, $v = (2\Delta P/\rho)^{1/2}$ so that $Re = Lv\rho / \mu = L (2\Delta P\rho)^{1/2} / \mu$.

For air: $Re = L (2 \times 5 \rightarrow 100 \text{ Pa} \times 1.225 \text{ kg/m}^3)^{1/2} / (1.8\times10^{-5} \text{ Pa s}) = 1.9 \rightarrow 8.7 \times10^5 \times L$

For water: $Re = L (2 \times 0.5 \rightarrow 50 \text{ kPa} \times 1{,}026 \text{ kg/m}^3)^{1/2} / (1.2\times10^{-3} \text{ Pa s}) = 8.4 \rightarrow 84 \times10^5 \times L$

ρ and μ values are from Lide (2001). The ranges just overlap indicating that close dynamical similarity of the two flows is likely, at least in a first-order approximation.

Close dynamical similarity of the flows implies that air's C and n should be applicable to water. The flood water would flow through the same cracks which the air flowed through under similar fluid conditions. Since n represents the flow conditions, which are the same for air and water, no adjustment is needed. C, though, represents the fluid in that $C = C_{drag}A(2/\rho)^n$. The values of C must be adjusted so that $C_{water} = C_{air} (\rho_{air}/\rho_{water})^n$.

Concerns are detailed below about the transferability of C and n values from air to water.

Concern: Air and water are different fluids.

The flows are similar, thus the type of Newtonian fluid should not matter except in special circumstances. For example, air is compressible whereas water is incompressible, but the crack flow equations for air assume fluid incompressibility. Surface tension is a concern for the start of water flow through a crack. In the absence of air bubbles, surface tension would

not affect flow in progress. The calculations here may assume that water instantaneously penetrates through a crack, i.e. surface tension effects are not considered. The major density difference between water and air is factored in by recalculating C. One other major assumption related to the fluid is that the flood water is free of debris and contaminants.

Concern: ΔP for air is 5-100 Pa but ΔP for water is 0.5-50 kPa.

This difference was considered in calculating Re, but the result was still that the air and water flows likely have close dynamical similarity.

Concern: C and n values are for simultaneous flow of air in and out of a structure whereas water would first enter the residence and then there might be flow through.

The data for air in Orme *et al.* (1998) purport to give flow rates through each crack, so the ultimate pathway of the fluid and the direction of flow should not be significant. Additionally, as soon as water enters a residence, it theoretically would be flowing through. Simultaneous flow in and out would occur even when the net flow rate is into the residence. Thus, a reasonable assumption is that this factor does not impact first-order calculations.

Concern: The air flow calculations assume that Q_{crack} is invariant with y and t which is clearly not the case for water. For water, ΔP is a function of y and would decrease over time due to the rising water inside the residence.

In the calculations, Q_{crack} is integrated with respect to y with ΔP as a function of y. The approximation that crack geometry varies with neither x nor y is reasonable for first-order calculations. C values for air are provided per crack length and per gap area indicating that they are averaged over crack geometry. While Q_{crack} could also be integrated with respect to t with ΔP as a function of t, the calculations here will seek indicative and maximum values of FRR. An instantaneous Q_{crack} at $t = 0$ with maximum f_{diff} will be sought rather than a total Q_{crack} through time until $f_{diff} = 0$.

Concern: n varies with ΔP.

The variance of n with ΔP is complex; for example, provided by Sherman (1992) for cylindrical pipe flow:

$$n = \tfrac{1}{2} \left[1 + \left(1 + \left(\text{constant} \right) \times \left(\text{pipe area} / \text{pipe length} \right)^2 \times \Delta P / \left(32\pi^2 \rho v^2 \right) \right)^{-1/2} \right]$$

The presence of the ratio $\Delta P/\rho$ means that the variation of n with ΔP would be similar for air and water in this case. For cracks, air flow calculations approximate that n is invariant with ΔP, hence water calculations may do so too at the first order.

8.1.3 Past Studies: Quadratic Formula

The main alternative to the power law is the quadratic formula $\Delta P = \alpha Q_{crack} + \beta Q_{crack}^2$ which yields:

$$Q_{crack} = [-\alpha + (\alpha^2 + 4\beta\Delta P)^{1/2}] / (2\beta)$$

$Q \geq 0$ so the positive root must be selected.

Some studies (Orme *et al.*, 1998; Walker *et al.*, 1997) conclude that the power law is more realistic than the quadratic formula. Other studies (Baker *et al.*, 1987; Etheridge, 1998) disagree, stating that the quadratic formula is the proper route. A principal objection by Etheridge (1998) is that the power law is not valid at low ΔP, but this situation does not apply to flood water flow.

A second objection by Etheridge (1998) is that the power law is derived empirically through curve-fitting, whereas the quadratic formula is derived theoretically. This statement is questionable because:

- Sherman (1992) develops the quadratic formula for cylindrical pipes and is referenced by Etheridge (1998) as the source for this equation. Etheridge (1998) assumes, without explanation, that the same theory holds for all crack geometries found in residences. As well, Sherman (1992) states that the quadratic formula is obtained by "linearizing the Navier-Stokes equation [sic]", clearly an approximation backed up by observation. These two assumptions are acceptable, but then the theoretical derivation of the quadratic formula promoted by Etheridge (1998) is not entirely theoretical, instead suffering from the same criticisms applied to the power law.
- One of the factors in Etheridge's (1998) full quadratic formula is determined empirically (Sherman, 1992). This factor's value varies depending on the approximation applied for the calculations.
- Etheridge (1998) states "The quadratic equation was derived for *developing* flows and it does not rely on any significant length of fully developed flow within the opening; only that the flow at the outlet is fully developed. Even when the latter condition is not fulfilled, the quadratic equation can still be expected to be reasonably accurate". The power law, however, does not rely on such assumptions, instead assuming that the flow is never fully developed. Stating that "the quadratic equation can still be expected to be reasonably accurate" suggests that the quadratic formula may be

accurate enough for many purposes—different from claiming that the equation is indeed accurate under such circumstances.

- Etheridge (1998) uses Baker *et al.* (1987) to justify the use of the quadratic formula based on empirical data. Baker *et al.* (1987) support the use of the quadratic formula but accept its validity based on empirical tests whereas Etheridge (1998) seeks a theoretical justification. Baker *et al.* (1987) further justify the equation's use with theory based on flow through infinite parallel plates, an obvious (but acceptable) approximation from crack flow.
- One of Baker *et al.*'s (1987) coefficients is initially calculated by using both theoretical and empirical inputs and then, after their experiments, they suggest that "adequate estimates of the parameters can be readily obtained by quadratic regression analysis of the experimental data". Their approach is acceptable, but does not support Etheridge's (1998) case for a law based on theory.
- Baker *et al.* (1987) point out that "Although the quadratic disregards the existence of a critical velocity of transition between streamline and turbulent flow, it has the practical advantage that at both extremes…it gives the correct forms". Because crack flow is mentioned as being transitional between laminar and turbulent flow (Orme *et al.*, 1998), the quadratic formula may not apply.

A method for converting C and n into α and β is given by Etheridge (1998), which would make it possible to compare results from both formulae. The use of this conversion does not make the quadratic formula any more suitable than the power law because:

- If C and n are determined based on curve-fitting to the power law which does not apply to all situations, then determining α and β from C and n could not increase accuracy. The comparison assumes that at $\Delta P = 30$ Pa, the slopes and flow rates of both equations are equal. If the power law equation is erroneous, it may not be accurate enough at this reference pressure.
- The relationship between C and n values and α and β values is given as a function of ΔP, but ΔP is arbitrarily chosen as 30 Pa. Both the power law and the quadratic formula could be applied for pressures well above the 50 Pa limit assumed.
- Etheridge (1977) criticises the power law for not being dimensionally homogeneous, but deriving α and β from C and n would create a similar problem for the quadratic formula.

8.1.4 Choosing the Preferred Equation

The power law is chosen for FRR calculations. First, the power law and the papers supporting it are adequately credible. Second, the power law is currently accepted more in practice than the quadratic formula. Third, data compiled from extensive empirical sources are available for C and n (Orme *et al.*, 1998) but α and β would need to be calculated from C and n. Introducing this extra step would not be useful.

8.2 Calculations

8.2.1 Assumptions

As noted in Section 8.1.2, only instantaneous FRR at t = 0 is calculated here. t = 0 is the most appropriate time because it yields FRR_{max} (unless v increases rapidly and substantially). The following assumptions must be made:

- No infiltration occurs prior to t = 0.
- The flood water reaches f_{diff} and v instantaneously.
- Any infiltrating water instantaneously spreads evenly over the plan area of the entire residence. Therefore, FRR = FIR / A.
- The flood water has neither debris nor contaminants.
- Only f_{diff} up to the top of the ground floor is considered. The reason will become evident in Chapters 9, 10, and 11, but was hinted at in Chapter 7 in suggesting that if f_{diff} reaches that level, the equivalent of the residence's complete destruction (DS5) would be likely.
- v is constant over the face of the structure which v impacts.

One further consideration is that, unlike f_{diff}, v cannot impact all residence walls simultaneously. Nonetheless, assuming this impossible situation would establish an upper bound for FRR_{max}, hence this approach is adopted. If the upper bound is not useful, then this assumption would need to be revisited.

Other assumptions relate to residence components and are provided in Section 8.3.

The basic pressure equation, from which the formulae are derived, is:

$\Delta P = \rho_w g(f_{diff} - y) + \frac{1}{2}\rho_w v^2 = \Delta P_{y=0} - \rho_w g y$ for $b \le y \le f_{diff}$

$\Delta P = 0$ for $y > f_{diff}$

8.2.2 Infiltration Through Cracks: Length Infiltration

The relevant residence components for length infiltration through cracks are doors, windows, door and window frames, and the joints between other openings and the wall. Infiltration would occur along the bottom crack, the two side cracks, and the top crack. Wall perimeter interfaces with floors and ceilings are length infiltration too, but only along horizontal lengths. Wall-wall interfaces are included in wall area infiltration (Section 8.2.3). The equations are developed from $Q_{crack} = C\Delta P^n$ by integrating along each component.

$$Q \text{ for bottom crack} = C\,(\Delta P_{y=b})^n\, W = CW \times (\Delta P_{y=0} - \rho_w g b)^n$$

$$Q \text{ for side crack if } b \le f_{diff} < s = C \int_b^{f_{diff}} (\Delta P_{y=0} - \rho_w g y)^n \, dy$$
$$= C / [\,(\rho_w g(n+1)\,] \times [(\Delta P_{y=0} - \rho_w g b)^{n+1} - (\Delta P_{y=0} - \rho_w g f_{diff})^{n+1}\,]$$

$$Q \text{ for side crack if } f_{diff} \ge s = C \int_b^{s} (\Delta P_{y=b} - \rho_w g y)^n \, dy$$
$$= C / [\,(\rho_w g(n+1)\,] \times [(\Delta P_{y=0} - \rho_w g b)^{n+1} - (\Delta P_{y=0} - \rho_w g s)^{n+1}\,]$$

$$Q \text{ for top crack} = C\,(\Delta P_{y=s})^n\, W = CW \times (\Delta P_{y=0} - \rho_w g s)^n$$

8.2.3 Infiltration Through Cracks: Area Infiltration

The relevant residence components for area infiltration through cracks are walls. The equation is developed from $Q_{crack} = C\Delta P^n$ by integrating over the wall's area, but not including the portion of the wall below ground floor level. Water infiltrating through this part of the wall would go underneath the floor.

$$Q \text{ for area if } b \le f_{diff} < s = CW \int_b^{f_{diff}} (\Delta P_{y=0} - \rho_w g y)^n \, dy$$
$$= CW / [\,(\rho_w g(n+1)\,] \times [(\Delta P_{y=0} - \rho_w g b)^{n+1} - (\Delta P_{y=0} - \rho_w g f_{diff})^{n+1}\,]$$

$$Q \text{ for area if } f_{diff} \geq s = CW \int_b^s (\Delta P_{y=b} - \rho_w g y)^n \, dy$$
$$= CW / [(\rho_w g(n+1)] \times [(\Delta P_{y=0} - \rho_w g b)^{n+1} - (\Delta P_{y=0} - \rho_w g s)^{n+1}]$$

v impacts only the outer leaf of a cavity wall whereas infiltration must also pass through the inner leaf. Therefore, it is unlikely that velocity-induced infiltration could occur through a cavity wall. In case this assumption is inappropriate and in order to establish an upper bound for FRR, calculations will still be completed for wall infiltration including ΔP from v. If the upper bound is not useful, then this assumption would need to be revisited.

8.2.4 Infiltration Through Orifices

The relevant residence components for infiltration through orifices are postal flaps and other openings such as air bricks, ducts, pipes, and vents—but not the joints of these openings with the wall because they were considered for length infiltration (Section 8.2.2). The nominal equation is the standard orifice flow equation (e.g. Franzini and Finnemore, 1997; Orme *et al.*, 1998) which is the general power law (Section 8.1.2) with n = ½:

$$Q_{orifice} = C_{drag} R (2 \Delta P / \rho_w)^{1/2}$$
$$= 2^{1/2} C_{drag} W \int_b^s [(\Delta P_{y=0} - \rho_w g y) / \rho_w]^{1/2} \, dy$$
$$= 2^{3/2} C_{drag} W / (3g) \times [(\Delta P_{y=0} / \rho_w - g b)^{3/2} - (\Delta P_{y=0} / \rho_w - g s)^{3/2}]$$

C_{drag} is dimensionless here because n = ½. Only $f_{diff} \geq s$ is considered because H for these openings tends to be small enough to traverse the 0.5 m interval considered for f_{diff} in these calculations.

8.3 Parameters

A (residence plan area) ∈ {38 m², 55 m², 84 m²} (Chapter 6).

C and n values for the power law equation are from Orme *et al.* (1998). The median value is used apart from one exception noted below.

C_{drag} (orifice drag coefficient) = 0.7 for postal flaps and 0.9 for other openings.

The lowest value is approximately 0.61 for a thin orifice with a sharp or square edge and the highest value approaches 1.00 for an orifice with a more rounded edge or longer path (Franzini and Finnemore, 1997; Orme *et al.*, 1998). Postal flaps present sharp, almost square edges, so C_{drag} = 0.7 is estimated. Other openings such as vents and pipes involve flow through the thickness of a cavity wall and may have rounded edges, so C_{drag} = 0.9 is estimated. FRR sensitivity to these choices is checked in Section 8.4.3.

g (gravitational acceleration) = 9.80665 m/s^2 (Lide, 2001).

Π (external residence perimeter) $\in$ {12 m, 25 m, 41 m} (Chapter 6).

ρ_w (water density) = 1,026 kg/m^3 (Lide, 2001).

Unless otherwise noted, data provided below on residence components have been determined from the residence survey. Decisions are also based on interviews with architects and engineers including Campbell (2001-2), McEvoy (2001-2), and Spence (2001-2). Because results reported here are for $f_{diff} \leq 2.5$ m which is for inundation of the ground floor only, b and s may be set exactly for residence components.

Doors and door frames:

- All hinged doors have H = 2.0 m, W = 0.8 m, and b = 0.3 m.
- All sliding doors have H = 2.0 m, W = 2.0 m, and b = 0.3 m with four side cracks due to the extra two in the middle between the two panels.
- Crack geometry is assumed to be the same around the entire perimeter.
- Postal flaps have W = 0.20 m and H = 0.05 m and no other openings occur on doors except for windows which are considered under windows.

Floors and Ceilings:

- Water infiltration does not occur through concrete floors (Bramley and Bowker, 2002) but does occur through the wall perimeter interface with the concrete floor.
- The wall perimeter interface with the ground level floor is either concrete or timber depending on the floor type.
- The wall perimeter interface with floors and ceilings on upper levels is timber.

- For the wall perimeter interface with floors and ceilings, Orme *et al.* (1998) write:

  > It is important to undertake a survey of visible gaps and cracks. Performance is influenced by age during which time openings occur through general movement and settling. It is suggested that upper quartile values should be considered for buildings of greater than 2-3 years, although this has yet to be verified.

  The upper quartile is taken because most residences surveyed are at least 2-3 years old and to ensure that the results reported here are robust through time.

Openings (air bricks, ducts, pipes, postal flaps, vents, and other small openings):

- Orme *et al.* (1998) write that "Data should only be applied to components that penetrate the inner leaf of the building". Air bricks are excluded based on interviews with architects, comments in Chudley (1995) and DTLR (2002), and information from British firms (e.g. Timloc, Cullen Building Products, and Support Site Ltd.). Modern cavity walls are ventilated with air bricks and small plastic inserts in the outer leaf, although the inserts' primary function is usually draining water from the inside of the outer leaf. These openings rarely penetrate through to the living areas of the residence, although it was common in older residences for boilers and larders to be ventilated to the outside. In the modern residences in the case study sites, air bricks typically lead through to underneath a suspended floor while the plastic inserts ventilate only the cavity. Flood water cannot enter the living space of the residence directly through these routes. Instead, water would enter the cavity and then percolate through the walls or else fill up the space underneath the floor and then flow up through the floor. The limiting elements in these pathways' flow rates are infiltration through the blockwork inner leaf and the suspended floor respectively. Therefore, the specific rate of flow through air bricks or plastic inserts does not factor directly into the FRR calculations and these items are not included in the FRR calculations. This approach establishes an upper bound for FRR because of the assumption that air bricks and plastic inserts do not retard flow into the wall cavity or into the space underneath the floor. If the upper bound is not useful, then this assumption would need to be revisited.
- Both the joint between the opening and the wall and the opening itself must be considered. C and n coefficients from Orme *et al.* (1998) are provided for the joints.

For the opening, the orifice flow equation is used. The joints and the openings would be sealable separately, so it would be possible to have infiltration through the joints but not the openings and vice versa.

Walls:

- Only cavity walls are considered (Chapter 6). Data in Orme *et al.* (1998) are for cavity walls on the assumption that the inner leaf is the best barrier and thus dominates FIR.
- With respect to finish, infiltration data exist for timber panel, wall boards, cladding, and concrete blocks (Orme *et al.*, 1998). Parts of 11% of walls in Canvey Island and 0% of walls in Hull are covered by these materials which is small enough coverage that they do not need to be considered. Infiltration data also exist for rendered walls and unrendered walls which are considered separately.
- Infiltration through interfaces between walls is included in the data for wall area (Orme *et al.*, 1998).
- The area of windows, doors, and openings must be subtracted from $R_{wall} = H_{wall} \times W_{wall}$ to determine the wall area through which infiltration occurs. A constant wall fraction not devoted to openings of 0.80 is assumed. This fraction varies with y since doors reach the base of walls whereas windows tend to be at middle heights, but such variation is not considered. FRR sensitivity to the choice of 0.80 is checked in Section 8.4.3.

Windows:

- Crack geometry is assumed to be the same around the entire perimeter.
- To calculate infiltration through windows, all side lengths of all individual panes in all units must be identified. Vertical and horizontal side lengths must be differentiated and b and s for each side length is needed. As an initial assumption, all windows were assumed to have the equivalent of H = W = b = 1.0 m.

8.4 Results

8.4.1 FRR By Residence Component

Table 8.4.1.1 compares FRR by residence component for a typical range of f_{diff} and v.

Table 8.4.1.1: Order of Magnitude for FRR (m/s) For Single Components

Component	Approximate FRR (m/s) considering $f_{diff} \leq 2.5$ m (one storey) and $v \leq 5.0$ m/s
door, hinged, non-weather-stripped (including frame)	10^{-4}
door, sliding, non-weather-stripped (including frame)	10^{-4}
door, hinged, weather-stripped (including frame)	10^{-4}
door, sliding, weather-stripped (including frame)	10^{-5}
floor, suspended timber	10^{-3}
opening, discharge pipe (including joint)	10^{-5}
opening, discharge pipe, joint only	10^{-5}
opening, pipe (including joint)	10^{-5}
opening, pipe, joint only	10^{-5}
opening, postal flap (including joint)	10^{-3}
opening, postal flap, joint only	10^{-5}
opening, sealed spiral duct (including joint)	10^{-3}
opening, sealed spiral duct, joint only	10^{-5}
opening, vent (including joint)	10^{-3}
opening, vent, joint only	10^{-4}
wall perimeter, rendered	10^{-4}
wall perimeter, unrendered	10^{-4}
wall and floor/ceiling interfaces (timber ground floor)	10^{-3}
wall and floor/ceiling interfaces (concrete ground floor)	10^{-4}
window, hinged, non-weather-stripped (including frame)	10^{-4}
window, sliding, non-weather-stripped (including frame)	10^{-4}
window, hinged, weather-stripped (including frame)	10^{-5}
window, sliding, weather-stripped (including frame)	10^{-5}

The order for residence components from least to most contribution to FRR per component is:

1. pipe
   window, weather-stripped
2. concrete floor interface with wall perimeter
   door
   window, non-weather-stripped
   wall perimeter

3. duct
   postal flap
   vent
4. suspended timber floor (including interface with wall perimeter)

The numbers and properties of components affect their ultimate influence on FRR:

- Vents and ducts appear high up, with $b \geq 2.0$ m.
- Residences tend to have more windows than doors and tend to have few other openings.
- Residences tend to have only one postal flap.

Therefore, the component which contributes most to FRR is the presence of a suspended timber floor followed by the presence of a postal flap. Walls, windows, and doors would be approximately equivalent. Vents and ducts would contribute significantly to FRR for floods inundating the entire ground floor or for residences which have these components low down.

8.4.2 FRR by Residence Type

Estimating from Table 8.4.1.1, FRR would likely be less than 0.010 m/s for any residence when f_{diff} inundates only the ground floor. This value is instantaneous FRR at $t = 0$ representing the maximum possible FRR (unless v increases substantially to overcome the decrease of f_{diff} with time).

In order to confirm that this range is valid, a residence which would maximise FRR was designed. Doors and windows were assumed to be non-weather-stripped, walls were assumed to be unrendered, and a timber floor was assumed along with a postal flap. The ground floor walls were given $b = 0$ m while overestimations of the numbers of openings were made (Table 8.4.2.1).

Additionally, Π and A were selected to maximise FRR. As Π increases, flood water has a greater external perimeter by which to enter the residence. The number of openings increases too. Thus, when Π increases, FRR increases. In contrast, when A increases, FRR decreases because FRR = FIR / A. To maximise FRR, the largest Π of 41 m and the smallest A of 38 m^2 were selected. This residence type was not observed in the case study sites, but is considered to establish FRR_{max}.

FRR results are in Table 8.4.2.2. Tables 8.4.2.3 and 8.4.2.4 provide similar information for a more typical case based on the residence survey.

Table 8.4.2.1: Residence Design Which Maximises FRR

	Number of Ground Floor Components
doors, hinged, non-weather-stripped	4
doors, sliding, non-weather-stripped	4
openings, discharge pipes	5
Openings, pipes	5
openings, postal flaps	1
openings, sealed spiral ducts	3
openings, vents	3
windows, hinged, non-weather-stripped	10
windows, sliding, non-weather-stripped	10
suspended timber floor	
walls without finish	
b = 0.0 m for walls and doors	
A = 38 m^2	
Π = 41 m	

Table 8.4.2.2: FRR (m/s) for Residence Design Which Maximises FRR

		f_{diff} (m)					
		0.0	0.5	1.0	1.5	2.0	2.5
v (m/s)	0.0	0.000	0.003	0.006	0.010	0.025	0.036
	0.5	0.000	0.003	0.006	0.011	0.025	0.037
	1.0	0.000	0.003	0.006	0.011	0.026	0.037
	1.5	0.000	0.003	0.007	0.011	0.027	0.038
	2.0	0.000	0.004	0.007	0.012	0.029	0.040
	2.5	0.000	0.004	0.008	0.013	0.031	0.042
	3.0	0.000	0.005	0.009	0.014	0.033	0.044
	3.5	0.000	0.005	0.009	0.015	0.035	0.047
	4.0	0.000	0.006	0.010	0.016	0.038	0.050
	4.5	0.000	0.006	0.011	0.017	0.041	0.053
	5.0	0.000	0.007	0.012	0.018	0.044	0.056

Table 8.4.2.3: Typical Residence

(Still Designed for a Larger FRR)

	Number of Ground Floor Components
doors, hinged, non-weather-stripped	2
doors, sliding, non-weather-stripped	2
openings, discharge pipes	2
openings, pipes	2
openings, postal flaps	1
openings, sealed spiral ducts	1
openings, vents	1
windows, hinged, non-weather-stripped	5
windows, sliding, non-weather-stripped	5
suspended timber floor	
walls without finish	
b = 0.3 m for walls and doors	
A = 55 m^2	
Π = 25 m	

Table 8.4.2.4: FRR (m/s) for a Typical Residence

		f_{diff} (m)					
		0.0	0.5	1.0	1.5	2.0	2.5
v (m/s)	0.0	0.000	0.001	0.002	0.004	0.007	0.011
	0.5	0.000	0.001	0.002	0.004	0.007	0.011
	1.0	0.000	0.001	0.002	0.004	0.007	0.011
	1.5	0.000	0.001	0.002	0.004	0.008	0.011
	2.0	0.000	0.001	0.003	0.005	0.008	0.012
	2.5	0.000	0.001	0.003	0.005	0.009	0.013
	3.0	0.000	0.002	0.003	0.005	0.010	0.013
	3.5	0.000	0.002	0.003	0.006	0.011	0.014
	4.0	0.000	0.002	0.004	0.006	0.012	0.015
	4.5	0.000	0.003	0.004	0.007	0.012	0.016
	5.0	0.000	0.003	0.005	0.007	0.013	0.017

While FRR is rapid for high f_{diff} and v in the unrealistic residence of Table 8.4.2.1 which maximises FRR, most expected circumstances yield FRR < 0.020 m/s. This limit is reached for deep floods only. The suggested typical value is FRR ≈ 0.005 m/s. This upper bound and typical value would apply to any residence in the case study sites.

8.4.3 Sensitivity Analysis

Table 8.4.3.1 uses the residence in Table 8.4.2.3 to illustrate the variance of FRR for the main estimated variables in the severe flood of f_{diff} = 2.5 m and v = 5.0 m/s. The estimated variables have little effect on FRR and alter neither the general limit of FRR < 0.020 m/s nor the typical value of FRR ≈ 0.005 m/s established in Section 8.4.2.

Table 8.4.3.1: Sensitivity of FRR for f_{diff} = 2.5 m and v = 5.0 m/s
(The residence in Table 8.4.2.3 is used.
The base FRR, for typical values of the variables, is 0.017 m/s)

Variable	Low Extreme	FRR (m/s)	High Extreme	FRR (m/s)
b	0.0 m	0.019	0.5 m	0.016
C_{drag}	0.61	0.016	1.00	0.018
wall fraction which is not openings	0.50	0.017	1.00	0.017

If the main contributing components to FRR were removed from the residence, the percent change in FRR by altering the variables in Table 8.4.3.1 would increase substantially. In this situation, FRR << 0.020 m/s and large relative variations in FRR would not affect subsequent analysis (see Section 8.5.2).

8.5 Discussion

8.5.1 Validating the Results

Literature on this topic was not found, thus quantitative validation cannot be completed. Validation of the main pathways by which infiltration occurs can be examined by using anecdotal evidence. The following quotations are from postings at floodforum.net (2002) by people whose properties were affected by floods:

- Ray Cullis: "Did the sewage enter by the doors, leaks, windows or did it enter via back pressure from your toilet installations".

  Mary Dhonau: "To answer Ray Cullis's question Every answer is Yes!!" [sic].
- Julie Heather: "The water just rose up through the floor".
- Victoria Heywood: "We were woken…by our next-door neighbour who, fortunately for us all, had popped down to the kitchen for a glass of water and noticed the first trickle under his door…Within an hour the whole ground floor was flooded to a depth of about 6 or 7 cm".

- Anne Bury: "The water came up through the floors long before it came in through the doors."

As well, Figures 8.6.1.1, 8.6.2.1, 8.6.2.2 provide examples of pathways by which flood water could enter or exit a residence.

These quotations and figures must be viewed with caution due to their anecdotal nature, but a pattern emerges that water does infiltrate houses by any pathway available. Floors appear to be a major source of infiltration, as predicted by the calculations here. Because neither f_{diff} nor v are provided for Victoria Heywood's case, and because the rate of water rise outside the residence is not known, her quantitative statement cannot be used further. The observations of sewage infiltrating and infiltration via toilets are discussed in Section 8.6.

8.5.2 Using the Results

Section 8.1.1 stated that knowing FRR is necessary for determining the range of possible pressure differentials imposed on residences. A reasonable maximum to consider for FRR is approximately 0.020 m/s and this value will continuously decrease with time, as water rises inside the residence. Some of the assumptions also ensured that the FRR calculation would yield an upper bound. At this upper limit for FRR, the water level inside the residence rises slowly, so it remains low in comparison to the flood depth outside the residence. Therefore, $f_{diff} \approx f$ over a duration of seconds or minutes and the range of possible ΔP imposed on residences is the maximum feasible for the given scenario. The upper bound assumptions were appropriate and do not need to be revisited because they provide a useful result.

Even if $f_{diff} = 0.5$ m, the minimum considered, ΔP due to f_{diff} would have several seconds over which to act to potentially break glass or walls before enough water is inside the residence to counteract the external ΔP. The FRR calculation illustrates that, under the assumption that f_{diff} and v are reached instantaneously, ΔP due to f_{diff} must be considered. Infiltration does not occur quickly enough to negate this external ΔP. The analysis of ΔP from f_{diff} and v breaking other residence components should proceed.

For influencing FRR, rendering walls makes little difference but sealing windows and doors and caulking their frames can have a minor impact. The presence or absence of suspended timber floors and postal flaps has the most impact. The rare case of a large opening such as a vent or duct low down could double FRR. Sealing such an opening, although not necessarily

sealing the joint, would significantly reduce FRR. These influences are not quantified in detail because they change neither the order of magnitude nor the typical value nor the upper bound of FRR. Likewise, for scenarios with the lowest FRR, namely FRR << 0.020 m/s, large relative variations are unimportant because they will not reduce ΔP imposed on the residence. Overall, specific residence characteristics do not alter FRR enough to require calculation for specific residences in the case study sites.

These calculations and results indicate how infiltration might be influenced but do not suggest that the goal is to reduce or prevent infiltration. This issue is discussed in Chapter 13.

8.6 Critique

8.6.1 Concerns with the Calculations

Weaknesses in the calculations include the direction of opening of postal flaps. When inundated, a postal flap which opens inwards would be expected to be pushed open by ΔP to permit water flow into the residence. In contrast, a postal flap which opens outwards could be pushed and held shut by ΔP. Alternatively, since postal flaps are generally made from plastic or light metal, they may be buoyant and easily opened by small magnitude or direction fluctuations in currents. As well, some observed outward-opening postal flaps were curved rather than flat, thereby preventing a tight seal from forming irrespective of ΔP keeping it shut. 59% of observed postal flaps opened outwards meaning that these ambiguities could affect FRR calculations for a significant proportion of residences. Although the overall conclusions would not be affected, specific FRR values for specific residences could be erroneous.

One of the upper bound assumptions was that neither air bricks nor plastic inserts in the outer leaf of a cavity wall affect FIR. If this explanation for infiltration through cavity walls is accepted—i.e., water infiltrates instantaneously through the outer leaf to fill the cavity and then infiltrates through the inner leaf and suspended floor into the residence—then rendering and other external finishes would not affect FIR. Orme *et al.* (1998) provide C and n values for external finishes on cavity walls without explaining the infiltration route through a cavity wall. Furthermore, v impacts only the external leaf. ΔP from v would contribute to filling the cavity but the pressure imparted may not contribute to infiltration through the inner leaf.

Although the overall conclusions would not be affected, specific FRR values for specific residences could be erroneous.

As well, turbulence in the flood would be expected in the vicinity of each residence. Such rapid fluctuations in v would affect the contribution to FRR from v. Suction effects may even produce $\Delta P < 0$, temporarily drawing water through air bricks out of the cavity or reducing the effective ΔP across components. The results presented here are an upper bound for FRR and apply instantaneously to $t = 0$. Assuming no suction effects is adequate.

The main concern with assuming that crack geometry does not vary around a component's perimeter is that the crack underneath a door tends to be larger than cracks at the sides and top. C and n are provided for only a door, not for each door crack. Etheridge (1977) provides some data for the lower crack in doors, but also indicates that crack geometry varies significantly along a crack, irrespective of crack location. Figure 8.6.1.1 illustrates flood water infiltrating through door cracks. The distinct streams could be a result of either debris blocking the water or crack geometry variation around the door. For the first-order results sought here, assuming invariant crack geometry around a component is nonetheless appropriate because differences are smoothed out by averaging.

Figure 8.6.1.1: Water Flowing Through Cracks Around St. Margaret's Church Door, King's Lynn, During the January 1978 Storm Surge (from Harland and Harland, 1980)

Trickle ventilators and the small plastic inserts in the outer leaf of cavity walls could not be identified from the residence survey, so FRR from them was not quantified. Finally, these calculations assume pure ocean water. Debris, contaminants, and solutes could significantly affect the physical properties input into the equations. They could also introduce unpredictable factors such as corroding or eroding cracks to increase crack area or clogging

cracks to decrease crack area. As described in floodforum.net (2002; see also Section 8.5.1), sewage does infiltrate residences but calculations for sewage are not explored here.

8.6.2 Concerns with the Model

Water exerts a relatively large ΔP on residence components. Distortion would be expected in components such as window frames and doors while any spaces in brickwork or blockwork would be rapidly exploited by infiltrating water. Figures 8.6.2.1 and 8.6.2.2 provide examples where the infiltration of water through a masonry wall is higher than would be predicted from the calculations here due to large holes providing a flow pathway.

Figure 8.6.2.1: Flooded Wine Cellar at Darwin College, Cambridge, U.K., October 2001
(The arrow identifies water flowing in through a hole in the masonry wall)

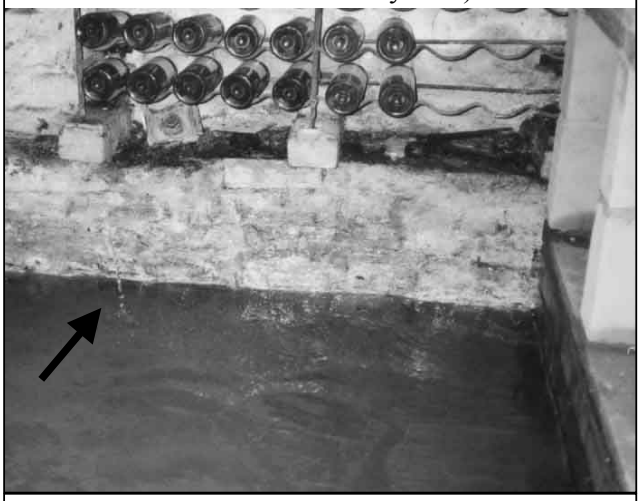


Figure 8.6.2.2: Water Flowing Through a Hole in a Brick in Malton, U.K., November 2000
(The basement on the other side of this wall was flooded)

Exploitation of paths through walls would generally increase FRR, but distortion could either increase or decrease FRR. Doors and windows which open outwards or which slide would be pushed into the residence, potentially sealing cracks while those which open inwards would be distorted such that the cracks would likely increase in size.

In the case study sites, few windows open inwards and few doors open outwards (Chapter 6). Because doors' b is 0.5-1.0 m less than windows' b, doors contribute more per component to FRR than windows. Residences, though, have more windows than doors. Quantifying the effect of distortion on FRR is subject to diverse, large uncertainties.

One other missing element in the FRR calculations is that flood water may be forced back through sewers and into residences through toilets and bath, shower, and sink drains (BRESL, 1996; DTLR, 2002; EA/CIRIA, 2001b; floodforum.net, 2002). For such infiltration to occur, a definitive connection between the external flood water and the outlet pipes must exist. Possible mechanisms are leaky pipes, which would be unlikely to afford much room for infiltration, or a combined storm and wastewater sewer system.

Canvey Island does not have a combined sewer system, but the wastewater system is from the 1930s and is leaky (Edwards, 2002). The areas studied in Hull, and most of the rest of the city, have a mixture of combined and separate sewer systems, at times with more than one system on the same street, even in the new developments (Hutchinson, 2002; Marr, 2002; Navas, 2002; Yorkshire Water, 2002). Predicting the significance of infiltration through the wastewater system is thus difficult without empirical evidence, but this pathway should not be ignored and could be anticipated and prevented by occupants. One other potential pathway for sewage infiltration is wastewater treatment or pumping facilities breaking down, possibly due to flooding, resulting in sewage pipes backing up or in overflow from the facilities.

Flood water may also enter a residence through groundwater rise, either water table rise or capillary action through soil, in the absence of ΔP from f_{diff} or v. This specific entry route was not considered in the FRR calculations. Water from groundwater rise would be under the same infiltration limitations as water from overland floods, such as the non-permeability of concrete floors. In many circumstances, groundwater rise would cause water to rise outside the residence too, not just into the space beneath a suspended floor. As the water rises outside a residence, $f_{diff} > 0$ may result and FRR calculations may be calculated similarly to the procedure outlined in this Chapter.

Questions also exist regarding infiltration through pipes and inlets for telephone and cable wires. Infiltration certainly occurs as crack flow around the joints of these openings with the wall. Unless the pipes or wires are torn away by the flood, or unless they are leaky, orifice infiltration could not occur. Some discharge pipes are open to the outside, so orifice infiltration would occur through them. Since these openings input minimally into FRR, the results are not affected. If highly accurate and precise FRR calculations were desired, this issue would need to be resolved.

Finally, the FRR calculations assume a typical but ideal residence with no external influences on infiltration characteristics. Factors resulting in deviations from the ideal such as cracks from damage, differential movement due to subsidence or heave, or poor construction or maintenance could substantially alter the calculated FRR.

8.7 Conclusions

A reasonable maximum for FRR is 0.020 m/s but a typical value is 0.005 m/s. These values are upper bounds. Even the maximum's upper bound is small enough to indicate that f_{diff} and v would impose significant ΔP on a residence. These results should be viewed with caution due to weaknesses in the calculations and the model used. Due to lack of sources, validation was not possible.

Nonetheless, the FRR analysis has validated the study of damage from f_{diff} and v due to ΔP. The FRR calculation may also indicate that sealing a residence to flood water will not necessarily minimise overall damage if other components are damaged by a large ΔP.

9. Wall Failure

9.1 Theoretical Justification

9.1.1 Introduction

This Chapter examines wall collapse due to a combined hydrostatic and hydrodynamic flood water pressure. Cavity walls represent more than 95% of external residence walls in the case study sites (Chapter 6), hence only these walls are considered. These walls comprise a brickwork outer leaf attached to a blockwork inner leaf by metal ties and are neither precompressed, nor prestressed, nor reinforced.

The failure analysis does not consider an entire external wall. External walls on residences are partitioned vertically by supported timber floors and horizontally by inner, partition walls. This section of a wall is termed a wall panel and is the component used for the failure analysis, for which H and W are discussed in Section 9.3.1.

9.1.2 Past Work: Empirical Evidence

The literature rarely details non-uniform loading. The case of blast loading has been studied (e.g. Alexander and Taylor, 1973), but peak pressures last milliseconds. The rate of flood water rise in a residence (Chapter 8) suggests that ΔP from f_{diff} and v would be expected to last at least several seconds, so blast loading results are not helpful.

USACE (1988) examined hydrostatic loading. Their experiment subjected three brick-veneer solid walls and two concrete-block solid walls to a lateral water load. The water was controlled, rising slowly at less than 0.6 m/h, in order to measure the induced deflections and the water levels at which structural integrity was lost and at which failure occurred. The conclusion was that the tested structures "can safely withstand about [0.9 m] of waterhead" (USACE, 1988). This conclusion perhaps indicates the origin of the 0.9 m depth presented as a definitive threshold in USACE (1998) and USACE (1997), as noted in Section 2.4.3.

In the U.K., EA/CIRIA (2001b) advise that no sealing should take place "more than one metre above the level of the ground surrounding the property". DTLR (2002) recommend:

> For most properties the external walls should have sufficient strength to retain flood depths up to 900mm (3 feet) above external ground level, with no water inside the building. However, the actual water level that can be safely retained by the external walls will depend upon the construction, age and condition of the walls.

These results are an estimation mainly based on expert engineering judgement (Bowker, 2002). The 0.9-1.0 m maximum for unreinforced masonry walls withstanding lateral hydrostatic pressure suggested by USACE (1988), EA/CIRIA (2001b), and DTLR (2002) translates to $\Delta P_{avg} \approx 5$ kPa.

Table 9.1.2.1 displays a sample of available empirical evidence for uniform loading. From these results, wall panels in the case study sites under a uniform lateral pressure would be expected to fail in the approximate range 1-10 kPa, with the lower end of this range more common than the upper end. Translating the steady state uniform load results into a near-steady state hydrostatic plus hydrodynamic pressure is not straightforward, but a rough estimate would be that failure would occur when the linear load's ΔP_{avg} approximately equals the uniform ΔP for failure of 1-10 kPa.

The research in Table 9.1.2.1 has weaknesses, particularly the laboratory results. The experiments do not consider realistic walls with two leaves and doors and windows of the sizes and positions observed in real residences. The experiments are indicative of the range of failure ΔP expected while those which use solid walls are realistic simulations of walls without openings. Additionally, the wall edge conditions in the laboratory are most frequently simply supported, which would be weaker than the continuous supports found for most ground floor wall panels in residences. Hence, walls in residences may be expected to be stronger than the literature suggests, although the expected effect of openings is not clear (see Section 9.6).

Ties between the cavity wall leaves must be considered. West and Hodgkinson (1975) inferred that solid ties transfer the load between leaves whereas butterfly ties do not. Their conclusion was based on one wall with each tie type and is refuted by the results from West *et al.* (1977) where no pattern emerges with respect to ties. Instead, cavity walls appear to behave more as single-leaf walls with the two leaves failing independently. Other observations in Table 9.1.2.1 of cavity wall failure pressure compared to single-leaf failure pressure suggest that a reasonable, conservative assumption is that, irrespective of how a leaf

or wall fails, load transfer between the leaves through the ties is negligible. This worst-case assumption means that the leaves fail independently—or, at least, they fail more independently than not—an assumption validated by observations. Possible failure mechanisms for cavity wall failure are not discussed in the literature, so Section 9.1.4 proposes possibilities for the leaves failing independently.

Damp proof courses appear to have minimal impact. West and Hodgkinson's (1975) conclusion that damp proof courses have a significant effect resulted from considering an unrealistic scenario in which the damp proof course is placed between the wall and the steel frame holding the wall. West *et al.* (1977) support West and Hodgkinson's (1975) conclusions but admit they arise from an impractical situation. West *et al.* (1977) also tested one wall each with a blue brick damp proof course and a permagrip damp proof course, stating that "the enhanced lateral resistance is marked" compared to walls without a damp proof course. This conclusion is not solidly based on the data considering that (a) the sample size was one each for walls with damp proof courses and (b) larger pressure variations were observed between two similar walls than between walls with and without a damp proof course.

The other references in Table 9.1.2.1 rarely mention that damp proof courses may be important and the values reported do not indicate any significant influence from them. Southcombe and Tapp (1988) suggest that for yield line analysis (Section 9.2) "a nominal degree of fixity at the dpc [damp proof course], derived from the self weight of the masonry corresponding to 10% of the mid-span moment" would be appropriate. The other authors neither support nor refute this contention.

The empirical research provides the failure pressures expected in order to validate analysis of failure under flood water pressures. To start such analysis, a failure mechanism must be postulated and an analysis technique is needed. For failure mechanism, BS 5628-1 : 1992—based on the work of, amongst others, West *et al.* (1977), Hendry (1981), and Hendry *et al.* (1981)—determines brickwork strength with tests which break through the bricks. This approach is surprising considering that the tensile strength of mortar is in most cases likely to be weaker than the brick, particularly for the residences in the case study sites (Campbell, 2001-2; Spence, 2001-2).

Empirical evidence indicates that failure under lateral pressures in brickwork and blockwork is characterised by failure along the mortar joints rather than through bricks. Laboratory work (CERAM, c. 2001; Sinha, 1980; Templeton *et al.*, 1988a and 1988b; USACE, 1988), in situ tests (Cavanagh *et al.*, 1982; Spence and Coburn 1992), and observed failure incidents (Haseltine *et al.*, 1977; Figures 9.1.2.1, 9.1.2.2, and 9.1.2.3) illustrate the tendency for failure along mortar joints with masonry units occasionally breaking. Selecting an analysis technique which incorporates failure along mortar joints is thus important.

Figure 9.1.2.1: Collapsed Brickwork Wall Along Queen's Road at the Back of St. John's College, Cambridge, U.K., October 2001
(The collapse was likely due to undermining when the depth of the water channel (on the right) increased during flooding.)

Figure 9.1.2.2: Brickwork Wall Hit by a Volvo Along Fen Causeway, Cambridge, U.K., December 2001

Figure 9.1.2.3: Brickwork Wall Broken From Behind by Flood Water, Malton, U.K., November 2000

Table 9.1.2.1: Examples of Empirical Research Into Wall Failure Under Uniform, Lateral Loading

Reference	Wall Type	H	W	Notes	Failure Pressure
Cavanagh *et al.* (1982)	Cavity walls with two brickwork leaves in an old building.	2.2 m	3.6 m	-damp-proof course -plaster on the inside -bricked-up opening	12.5 kPa
		2.2 m	4.57 m	-damp-proof course -plaster on the inside -boarded-up opening	12.5 kPa
CERAM (c. 2001)	22 single-leaf brickwork panels (laboratory testing).	2.1 m to 4.5 m	3.5 m to 5.5 m	-some with one side unrestrained -some with a central opening	lowest 0.95 kPa highest 7.2 kPa
	Fifteen leaf blockwork panels (laboratory testing).	2.1 m to 4.5 m	3.6 m to 5.5 m		lowest 0.25 kPa highest 2.6 kPa
Duarte (1998)	Sixteen single-leaf brickwork walls (laboratory testing).	1.2 m	1.2 m to 1.8 m	-each wall with a 0.4 m × 0.4 m opening, but the opening's position varied	lowest 2.6 kPa (W = 1.8 m) highest 10.2 kPa (W = 1.2 m)
Easterbrook (2001)	Single-leaf brickwork panel (laboratory testing).	2.6 m	5.5 m		2.6 kPa
	Single-leaf brickwork panel (laboratory testing).	2.6 m	5.5 m	-central opening with H = 1.4 m and W = 2.8 m	2.0 kPa
Gairns and Scrivener (1988)	Five single-leaf blockwork panels (laboratory testing).	2.58 m	2.35 m to 4.75 m		lowest 2.64 kPa (W = 4.75 m) highest 5.16 kPa (W = 2.35 m)
	Two single-leaf brickwork panels (laboratory testing).	2.58 m	4.75 m		3.63 kPa 3.82 kPa
Sinha (1980)	Two single-leaf brickwork panels (laboratory testing).	-H and W are not given, but "all test panels…were built in 1/3rd scale bricks" -each panel had a small central opening			8.31 kPa 10.34 kPa

Sinha *et al.* (1979)	Seven single-leaf brickwork panels (laboratory testing).	-H and W are not given, but "all the test walls were built with bricks one third of the normal size". -H/W ranged from 0.54 to 2.14.			least 1.37 kPa (H/W = 2.14) most 10.2 kPa (H/W = 0.54)
Southcombe *et al.* (1995)	Ten single-leaf brickwork panels (laboratory testing).	2.475 m	2.915 m or 5.615 m	-some with openings, but the opening's position varied	lowest 2.2 kPa highest 7.5 kPa
Southcombe and Tapp (1988)	Five single-leaf brickwork panels (laboratory testing).	2.475 m	4.840 m	-each wall had an opening, two similar to a window and three similar to a door	lowest 0.60 kPa highest 2.00 kPa
Templeton *et al.* (1988a)	A two-storey cavity wall house was built in a laboratory.	ground floor 3.150 m first floor 2.400 m	2 panels, 4.222 m each	-loading was applied starting at 0 kPa in 0.25 kPa increments	cracking started at 1.25 kPa but the house was still taking load when the testing was stopped at 3 kPa
West *et al.* (1977)	91 Single brickwork leaves (laboratory testing)	mainly 2.6 m; some 1.3 m or 3.6 m	1.52 m to 5.50 m	-some with a damp-proof course	lowest 1.72 kPa (W = 5.50 m) highest 15.51 kPa (W = 1.52 m)
	Nineteen cavity walls with one brickwork leaf and one leaf of either blockwork or brickwork (laboratory testing).	2.6 m	3.6 m to 5.5 m	-some with solid ties and some with butterfly ties, but no significant difference in failure pressure noted	lowest 2.41 kPa (W = 5.5 m with butterfly ties) highest 6.4 kPa (W = 5.5 m with quadruple solid ties)

9.1.3 Past Work: Analysis Techniques

The following analysis techniques appear in the literature:

- Arching analysis (arching-action theory) has been used for some time. McKee and Sevin (1959) applied it to the design of masonry panels for blast loading, such as from an atomic bomb. Southcombe and Tapp (1988) describe weaknesses in the theory such as "that the lateral deflection of the panel is small and that no appreciable reduction in the length of a panel due to thermal or moisture movements occurs".
- Elastic analysis (Seward, 1982) assumes full elastic behaviour for wall panels. Gairns and Scrivener (1988) use elastic plate analysis which they state "is justified due to the essentially linear load-deflection curves commonly reported up to first cracking where the first crack is horizontal and the panel is supported on four sides". Duarte (1998) effectively agrees, but in extending the discussion suggests that "Elastic theory is the method that sounds theoretically right but has failed in predicting the failure pressures due to the lack of an adequate strength of some types of panels after cracking". Templeton *et al.* (1988a) observe elastic behaviour in their structure (see Table 9.1.2.1) until cracking started.
- Finite element analysis (Easterbrook, 2001; Lee *et al.*, 1996; Ma and May, 1988; Martini, 1998; May and Ma, 1988; May *et al.*, 1988) has been used to examine wall panels with openings and various support conditions. As with other analyses, approximations are required and some software packages do not explicitly consider failure along the mortar joints, instead producing straight failure lines through bricks. No obvious advantage is gained from finite element analysis.
- Fracture line analysis (Sinha, 1978) is discussed with yield line analysis.
- Strip analysis assumes plasticity and involves "only the well known methods of calculation of moments in simply supported beams subjected to uniformly distributed and point loads" (Duarte, 1998). Since wall panels do not display full plastic behaviour and since the walls considered are not simply supported, this technique would likely be inappropriate. Despite the plastic assumption, Gairns and Scrivener (1988) note that this method is developed empirically from brickwork, hence "successful application to blockwork would be fortuitous".
- Yield line analysis, described next.

Yield line analysis was originally developed for structural concrete (e.g. Johnson, 1967; McGregor, 1992) and therefore assumed plasticity. The theory proposes that a panel under a load develops a plastic hinge in a region of high moment. This hinge resists the hinging moment, transferring any increase in the load to other regions which then yield and become part of the hinge. This region of the hinge is a yield line. When yield lines have formed across the panel dividing it into slabs, these slabs rotate plastically under the applied load and the panel collapses.

In criticising the use of yield line analysis for brickwork, Haseltine *et al.* (1977) note:

> Clearly a brick wall could not be expected to reach a constant moment and hold it since brickwork is a brittle material which will fail as soon as its resistance moment is reached; no moment, or very little, will be carried across the resultant crack.

Gairns and Scrivener (1988), Martini (1998), and Sinha (1978) support this view. Sinha (1978) develops a modified yield line analysis technique termed "fracture line analysis" which avoids the assumption of brickwork plasticity. Sinha's (1978) assumptions are:

> All deformations take place along the fracture lines only, and the individual parts of the slab rotate as rigid bodies. The load distribution is in accordance with the stiffness in the respective directions. The fracture lines develop only when the relevant strengths are reached in the two directions…Every portion of the panel into which it is divided by the fracture lines is in equilibrium under the action of external forces and reactions along the fracture lines and supports.

Sinha (1978), though, obtains brickwork properties through experiments which break through bricks rather than the observed fracturing along mortar joints. He also places his test specimens horizontally rather than the true vertical position of wall panels, thereby ignoring the panel's self-weight.

Lovegrove (1988) adapts the theoretical explanation of yield line analysis to be appropriate for unreinforced masonry. He considers the energy needed to produce a crack in order to develop the same yield line analysis equations, thereby bypassing assumptions of plastic hinges or elasticity. Martini (1998) also proposes an equivalent of a plastic moment. For a horizontal crack, he uses the overturning resisting moment due to vertical loads. For a vertical crack, he suggests that "moment transfer takes place through friction between horizontal surfaces at the bed joints between courses". The implementation of this approach is described in Section 9.2.4.

The main design advantage of yield line analysis is that it provides an upper bound for failure ΔP. One failure mechanism is identified from which a failure ΔP is determined, but another failure mechanism could exist which would provide a lower failure ΔP. Yield line analysis thus indicates the upper limit at which failure would definitely occur, subject to the uncertainties in the parameters used. The designer knows that if the failure ΔP calculated by yield line analysis were applied to the wall, failure must occur irrespective of the failure mechanism. This upper bound is useful for knowing the limit beyond which the wall cannot survive. Conversely, yield line analysis does not suggest a lower bound for failure. The lowest ΔP for which the wall might fail remains unknown.

Many authors in Table 9.1.2.1 compare the different analysis techniques with experimental work. The experiments do not involve sample sizes sufficient for determining the statistical significance of the experimental values compared to the theoretical values within appropriate confidence limits. Sample size problems in West *et al.* (1977) were described in Section 9.1.2. Southcombe *et al.* (1995) conclude that "The yield line approach…tended to over-estimate the flexural strength of brickwork panels, especially for panels with fixed edges" yet provide no statistical evidence to support their contention. One more example is Duarte's (1998) vague conclusion that "the yield line method, with all its shortcomings, seems to better predict the failure pressures" than other techniques without any statistical analysis to back up his claim. The best that can be said based on past studies is that the theoretical techniques all have problems, but generally produce values similar to those in Table 9.1.2.1.

9.1.4 Failure Mechanism for Cavity Walls

Cavity walls represent more than 95% of external residence walls in the case study sites (Chapter 6), hence only these walls are considered. The discussion so far has described wall failure up to the point where pressure breaks one leaf of the cavity wall. The most appropriate approach, based on empirical data from Table 9.1.2.1, would be to assume that the leaves fail independently. The result is a conservative estimate for failure load. This assumption is addressed in Sections 9.5.2 and 9.6 regarding the choice of the leaf thickness parameter. Although this approach is acceptable, a plausible failure mechanism should be postulated to ensure that a potential explanation exists for this choice.

The key is to demonstrate that an explanation exists for the ties transferring negligible load between the two leaves under the relatively large loads imposed by flood actions. The ties are set in the mortar of each leaf which may provide this explanation. Under large compressive loads, the ties may buckle or they may be pushed through the mortar in the leaf opposite the load. Under large tensile loads, the ties may be pulled out of the mortar. Therefore, ties potentially transfer negligible load under either compression or tension.

Two possible failure mechanisms for cavity walls must be explored (Figures 9.1.4.1 and 9.1.4.2). In Figure 9.1.4.1, water enters the cavity through the air bricks which are low and numerous (Chapter 6). The water rises rapidly in the cavity inducing ΔP across the inner leaf. The ties experience tensile loading and the inner leaf breaks. The water in the cavity rushes into the house leaving ΔP across the outer leaf. The outer leaf might or might not collapse.

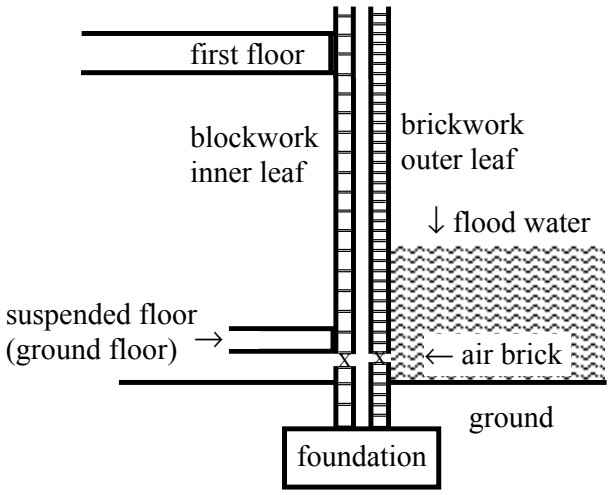
In Figure 9.1.4.2, water rises too quickly outside the residence for significant infiltration to occur through the air bricks or air bricks may not exist. ΔP exists across the outer leaf. The ties experience compressive loading and the outer leaf breaks. The outer leaf may collapse completely leaving ΔP exerted on the inner leaf which might or might not break. Alternatively, the cavity's narrow width may not permit the outer leaf to collapse completely. Instead, the load would be transferred to the inner leaf which might or might not collapse.

Figures 9.1.4.1 and 9.1.4.2 display the first floor, and by extension other floors above if they exist, as connected to only the inner leaf. This situation appears to be the most common (Addleson, 1989; Chudley, 1995). Previous discussion suggested that the ties transfer negligible load between leaves. Thus, it seems likely that the outer leaf would fail as a single panel over all storeys of the residence. When the outer leaf fails and the ties are under compression, however, they may buckle or they may be pushed through the inner leaf's mortar. If the ties buckle, then the outer leaf is liable to fail as a single panel. In this case, some of the slabs would span two storeys or more and it is unclear whether or not such a large slab would rotate as a rigid body, particularly when the pressure is experienced only at the bottom of the slab. If the ties are pushed through the mortar in the leaf opposite the load, then they would reach the first floor and likely be stopped (Figure 9.1.4.3). The ties may buckle or, at this stage, the panel below the first floor may start failing with a yield line forming near the first floor since an effectively continuous support exists there.

The situation is ambiguous. The literature, having considered neither non-uniform loads nor the cavity wall's mechanism of failure, does not discuss the failure mechanism to this level of detail. Therefore, for these calculations, the arbitrary assumption is made that the outer leaf fails with a yield line forming at the level of the first floor, as the inner leaf must. Section 9.6 illustrates that this assumption influences the results negligibly. Further research, most usefully laboratory work, would be needed to establish exactly how multi-storey, cavity wall, outer leaves fail and how ties influence the mechanism.

The mechanisms presented here are speculative but realistic. All failure pathways lead to severe structural damage representing DS4, possibly up to DS5, on the damage scale presented in Table 1.4.1. To verify these proposed failure mechanisms, laboratory work which examines each step of the failure process would be needed. Field observations of collapsed cavity walls may not leave clues enabling reconstruction of the pathway to failure. Despite the lack of either supporting or contrary evidence, the assumptions made in this Section are, at minimum, feasible. Thus, the calculations may proceed, but with caution warranted by the uncertainties. Further comment is provided in Section 9.6.

Legend for Figures 9.1.4.1 and 9.1.4.2



- The ground floor may be partially or fully embedded in the inner leaf.
- The ground floor may be solid rather than suspended.
- The foundation may be a raft or may extend farther than depicted.
- Some air bricks may be directly connected between the inner and outer leaves. Air bricks may be absent from Figure 9.1.4.2.
- In one-storey residences, the "first floor" would be the roof and the leaves would not extend above the roof.

Figure 9.1.4.1: Schematic for Wall Failure Where the Inner Leaf Fails First (Ties Under Tension)

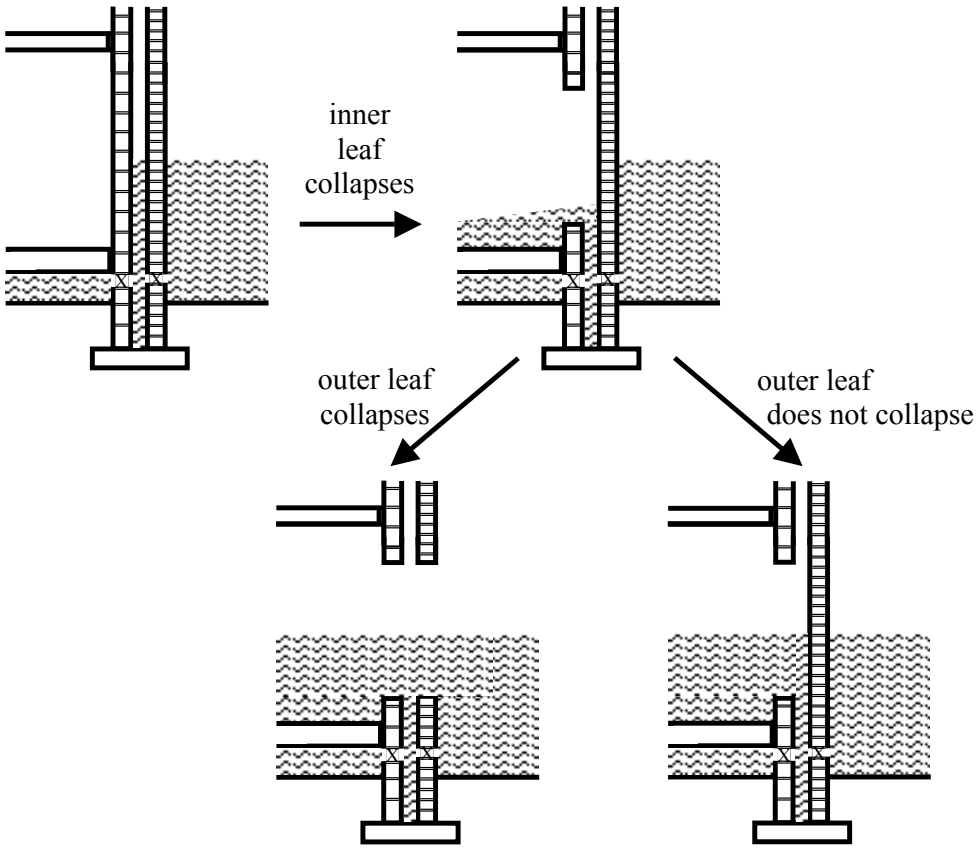


Figure 9.1.4.2: Schematic for Wall Failure Where the Outer Leaf Fails First (Ties Under Compression)

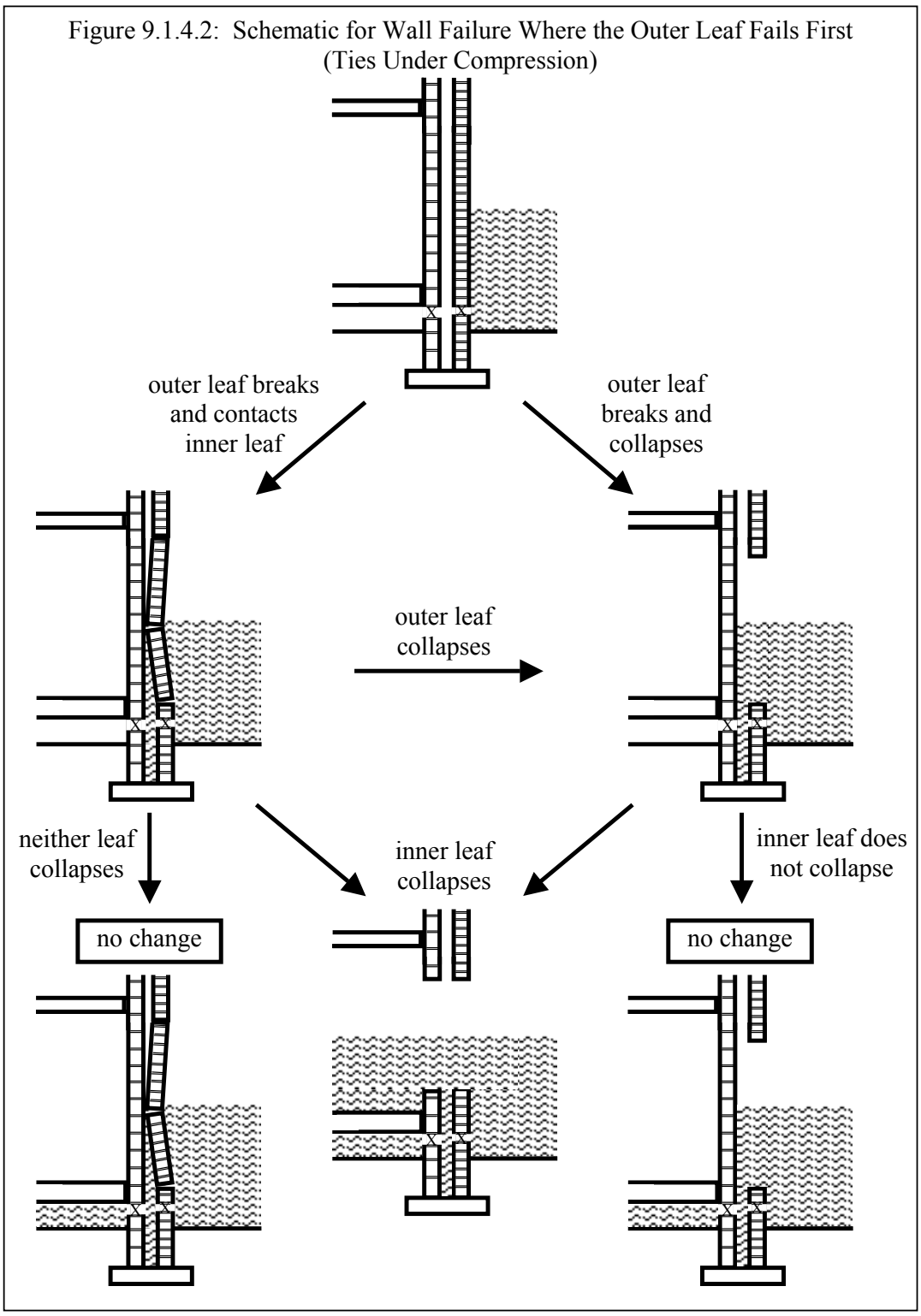
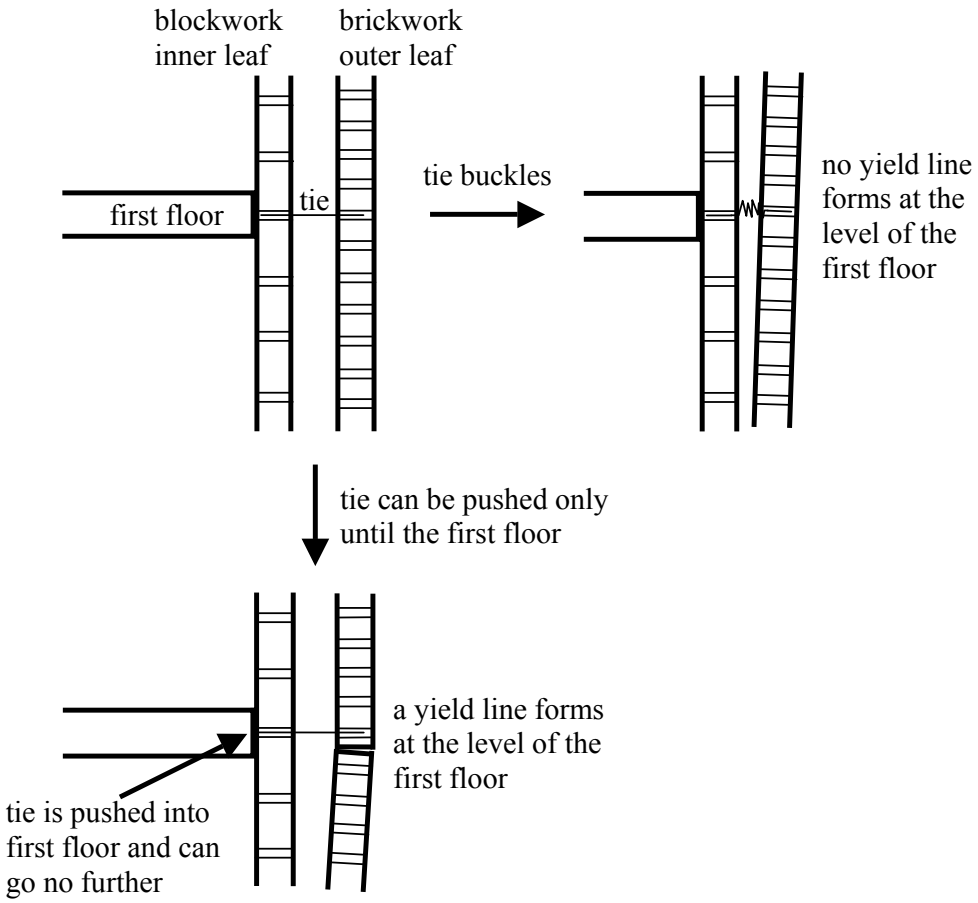


Figure 9.1.4.3: Potential Failure Mechanisms of the Brickwork Outer Leaf



9.1.5 Conclusions

Yield line analysis is by far the most studied and implemented technique, so it has been critiqued the most and developed the furthest. As well as this technique's sound basis in the literature, it permits feasible failure mechanisms, including failure along mortar joints as required from Section 9.1.2.

Therefore, for this study, yield line analysis based on Lovegrove (1988) and Martini (1998) is selected. The fundamental principle is the static equilibrium of an unreinforced masonry panel in the collapse limit state under external loads. The work method is adopted to define the equilibrium equations by analysing a small, arbitrary displacement δ of the slabs in the direction of the applied load. The method outlined in Section 9.2 draws upon many references from Section 9.1 other than Lovegrove (1988) and Martini (1998), but is tailored to the needs of this study, supported by Spence (2001-2), Ochsendorf (2001-2), and Morgenthal (2001-2). Few further references are given in Sections 9.2.

Yield line analysis is not a panacea, but is adequate for this study. The principal weaknesses are discussed further in Section 9.6.

Yield line analysis provides only the lowest ΔP at which the panel would definitely fail, not the highest ΔP at which the panel would definitely survive. Expected results for failure are that the average pressure on the panel would be in the range 1-10 kPa and most likely towards the lower end. From failure ΔP values, f_{diff}-v combinations which cause DS4 and DS5 from Table 1.4.1 may be calculated.

9.2 Calculations

9.2.1 Assumptions

The assumptions which must be made are:

- The failure mechanism described by modified yield line analysis is correct.
- Openings are not considered.
- Brickwork and blockwork are neither fully elastic nor fully plastic, but they are brittle.
- The yield line pattern which forms is shown in Figure 9.2.1.1 with idealised yield lines. This assumption is addressed in Section 9.6. Section 9.2.2 provides further explanation of this diagram and describes the real yield lines which occur.
- The slabs formed by the yield lines are rigid, i.e. they do not deform within themselves.

Additionally, an assumption illustrated in Figure 9.2.1.1 is that $H > f_{diff} > \varphi H$ at failure. The calculations and results include this situation plus the two other possibilities of $\varphi H > f_{diff} > 0$ and $f_{diff} > H$.

Figure 9.2.1.2 illustrates the basic pressure equation, from which the formulae are derived:

$\Delta P = \rho_w g(f_{diff} - y) + \frac{1}{2}\rho_w v^2 = \Delta P_{y=0} - \rho_w g y$ for $b \leq y \leq f_{diff}$

$\Delta P = 0$ for $y > f_{diff}$

Figure 9.2.1.1: Problem Definition for Wall Panel Failure

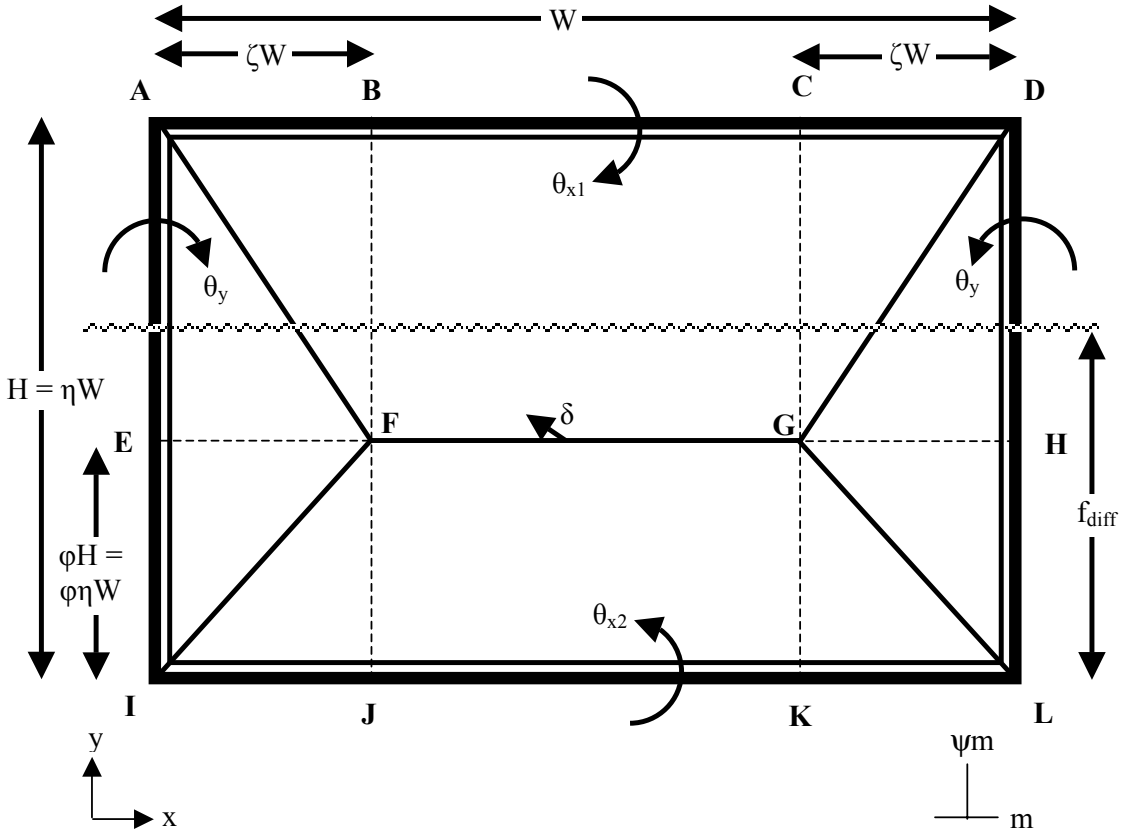
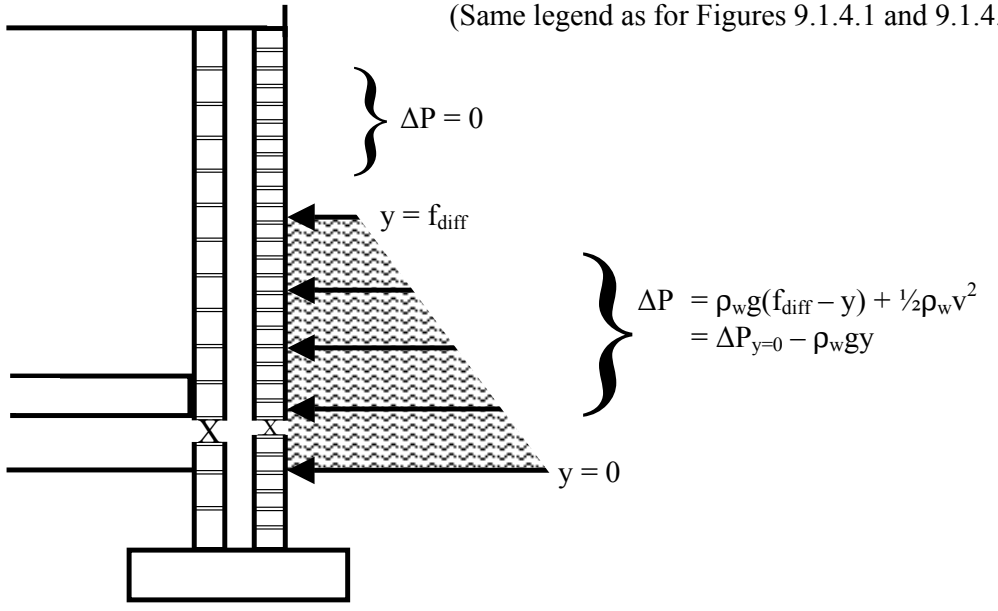


Figure 9.2.1.2: Pressure Function on a Wall
(Same legend as for Figures 9.1.4.1 and 9.1.4.2)



9.2.2 Definitions

The variables sought are f_{diff}, v, ζ, and φ. f_{diff} and v are determined via ΔP. By definition, $f_{diff}, v \geq 0$; $0 \leq \zeta \leq ½$; and $0 \leq \varphi \leq 1$.

m is a function of tension in the mortar and the weight supported by the wall panel above the yield line. A description of calculating m is in Section 9.2.4. ψ is a material property and is interpreted in Section 9.2.4.

H, W, and η are properties of the residences.

ρ_w and g are physical constants.

The edge conditions for **AI**, **DL**, and **IL** are set as continuous as the most realistic representation of residence walls. **AD** is also depicted as continuous in Figure 9.2.1.1, but would be simply supported in one-storey residences even for the leaf attached to the roof. The calculations in this Chapter assume that a yield line forms along **AD**, but where **AD** is simply supported and no yield line would form, the appropriate ultimate resisting moment term is set to zero. The yield lines are thus **AD**, **AF**, **AI**, **DG**, **DL**, **FG**, **FI**, **GL**, and **IL**.

Lines around which the slabs rotate are:

- **AD** (slab **ADGF** rotates θ_{x1});
- **AI** (slab **AFI** rotates θ_y);
- **DL** (slab **DGL** rotates θ_y); and
- **IL** (slab **FGLI** rotates θ_{x2}).

[x,y] Coordinates for nodes:

A [0 , ηW]	**B** [ζW , ηW]	**C** [W(1–ζ) , ηW]	**D** [W , ηW]
E [0 , $\varphi\eta$W]	**F** [ζW , $\varphi\eta$W]	**G** [W(1–ζ) , $\varphi\eta$W]	**H** [W , $\varphi\eta$W]
I [0 , 0]	**J** [ζW , 0]	**K** [W(1–ζ) , 0]	**L** [W , 0]

Lengths of Vertical Segments

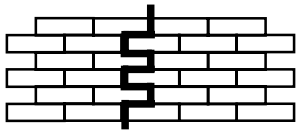
AI = **DL** = H = ηW

AE = **BF** = **CG** = **DH** = H(1–φ) = ηW(1–φ)

EI = **FJ** = **GK** = **HL** = φH = $\varphi\eta$W

Due to failure along the mortar joints rather than through the masonry units, the vertical yield lines actually comprise a sequence of horizontal and vertical segments (Figure 9.2.2.1).

Figure 9.2.2.1:
Vertical Yield Line in Brickwork and Blockwork.



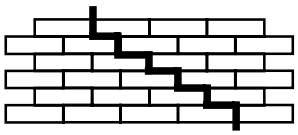
Lengths of Horizontal Segments

AD = **IL** = W

AB = **CD** = **EF** = **GH** = **IJ** = **KL** = ζW

BC = **FG** = **JK** = W(1 – 2ζ)

Figure 9.2.2.2:
Diagonal Yield Line in Brickwork and Blockwork.



Lengths of Diagonal Segments

AF = **DG** = $W\left[(1-\varphi)^2\eta^2 + \zeta^2 \right]^{1/2}$

FI = **GL** = $W\left[\varphi^2\eta^2 + \zeta^2 \right]^{1/2}$

Due to failure along the mortar joints rather than through the masonry units, the diagonal yield lines actually comprise a sequence of horizontal and vertical segments (Figure 9.2.2.2). No true diagonal yield lines exist in brickwork or blockwork.

Equation of **AF**: $y = -[(1-\varphi)\eta / \zeta] x + \eta W$

Equation of **DG**: $y = [(1-\varphi)\eta / \zeta] x + \eta W [1 - (1-\varphi) / \zeta]$

Equation of **FI**: $y = (\varphi\eta / \zeta) x$

Equation of **GL**: $y = -(\varphi\eta / \zeta) x + \varphi\eta W / \zeta$

Maximum deflection δ occurs along **FG** including the points **F** and **G**. Assuming that (a) $\tan\theta \approx \theta$ for small θ and (b) since length cannot change in the x-y plane, a gap appears between slabs (presumably due to the masonry's thickness):

δ = **BF** θ_{x1} = **FJ** θ_{x2} = **EF** θy

$\Rightarrow \eta W(1-\varphi)\,\theta_{x1} = \varphi\eta W\,\theta_{x2} = \zeta W\,\theta_y$

Therefore: $\theta_y = \eta(1-\varphi)\theta_{x1} / \zeta$

$\theta_{x2} = (1-\varphi)\,\theta_{x1} / \varphi$

$\delta = \eta W(1-\varphi)\,\theta_{x1}$

9.2.3 The Work Equation

Each slab **AFI**, **ADGF**, **DGL**, and **FGLI** must be in static equilibrium during the formation of yield lines. A balance between the work done by the external load and the work used

internally, or expended by the yield lines, is maintained. This balance is expressed by the work equation:

$$\sum_{\text{over all yield lines}} \text{m} \times (\text{length of yield line experiencing m}) \times \theta \text{ (slab's angle of rotation)}$$

$$=$$

$$\sum_{\text{over all slabs}} \int_{\text{over slab's area}} (\text{load experienced by a slab}) \times (\text{slab's deflection}) \, dx \, dy$$

9.2.4 The Work Equation's Left-Hand Side

m × (length of yield line experiencing m) is the ultimate resisting moment of a yield line which, at failure, equals the actual bending moment along that line. The resisting moment is a function of tension in the mortar and the weight supported by the slab above the yield line. For the slabs to rotate resulting in collapse of the panel, all yield lines would have formed, i.e. all mortar joints along yield lines would have broken and no tension would exist in the mortar along these lines.

For yield line analysis, Spence and Coburn (1992) assume that "The mortar used, whether of soil, cement or lime, is assumed to have zero tensile strength, but high compressive strength". Similarly, in developing a failure criterion for laterally loaded masonry walls, Guggisberg and Thürlimann (1991) write "the tensile strength of the mortar joints will be neglected". The assumption is thus made that this failure process of yield lines forming may be modelled by stating that all tension in the mortar joints is overcome to form the yield lines before the slabs begin to rotate.

Throughout this process, ΔP has not changed. f_{diff} and v are assumed to reach their values instantaneously and to be invariable during the time required for the yield lines to develop, the slabs to rotate, and the panel to fail—in the instances for which this sequence would occur. Although work is obviously done to break the mortar joints, this work does not enter the work equation because the slabs have not rotated. For the slabs to rotate, the resisting moment of m × (length of yield line experiencing m), based on either the weight supported by the slab above the yield line or friction between the rotating sections, must be overcome. Thus, the work equation compares this resisting moment with the moment applied by ΔP. If the latter is greater than the former, failure occurs. Otherwise, failure does not occur.

Calculating m × (length of yield line experiencing m) for horizontal components of yield lines involves only the weight above (in the y-direction) each yield line (see also Martini, 1998). Consequently, m × (length of yield line experiencing m) is a function of y incorporating the self-weight of the wall panel (minimum at y = s and maximum at y = b) and the dead and live loads of the structure which are transferred to this panel:

m × (length of yield line experiencing m)

= [(panel's weight above yield line) + (dead and live weight above yield line)]

× (distance of the weight's force vector from its line of rotation)

= [$\rho_b g \times$ (panel's volume above yield line) + $\gamma g W_{\text{yield line}}/W_{\text{panel}}$]

× (approximately $\eth C_f/2$)

= [$\rho_b g \eth \times$ (panel's area directly above yield line) + $\gamma g W_{\text{yield line}}/W_{\text{panel}}$] × $\eth C_f/2$

(C_f is the compression depth factor.)

Calculating m × (length of yield line experiencing m) for vertical components of yield lines involves a different mechanism. A yield line is theoretically a line with negligible thickness, hence the weight above a purely vertical yield line must be zero. Solely vertical yield lines cannot exist across a wall panel (see Figures 9.2.2.1 and 9.2.2.2). To factor in these complicated changes of direction, Martini (1998) postulates that, for the vertical components of yield lines, "moment transfer takes place through friction between horizontal surfaces". Spence (2001-2) notes that the work which must be expended to overcome the friction from the slab rotating around the yield line is calculated by:

$\psi(m_{\text{top of yield line}} + m_{\text{bottom of yield line}}) / 2 \times$ (vertical length of yield line) × θ

Using the mean of m is necessary since m changes linearly with y. ψ is interpreted as the friction coefficient between the two surfaces which rotate relative to each other, in this case masonry units and mortar.

For the left-hand side of the work equation, the resisting moment's work at failure in the yield lines is illustrated for segments **AD** (horizontal), **AF** (diagonal), and **AI** (vertical):

AD

$m_{y=\eta W} \times \mathbf{AD} \times \theta_{x1}$

= [$\rho_b g \eth \times$ (slab area directly above yield line) + $\gamma g W_{\text{yield line}} / W_{\text{panel}}$] × $\eth C_f / 2 \times \theta_{x1}$

= [$\rho_b g \eth \times 0 + \gamma g W / W$] × $\eth C_f / 2 \times \theta_{x1}$

= $\gamma g \theta_{x1} \eth C_f / 2$

AF

$$m_{y=\eta W(\varphi+1)/2} \times \mathbf{AB} \times \theta_{x1} + \psi m_{y=\eta W(\varphi+1)/2} \times \mathbf{AE} \times \theta_y$$

$$= [\ \rho_b g þ \times (\text{slab area directly above yield line}) + \gamma g W_{\text{yield line}} / W_{\text{panel}}\ ] \times þ C_f / 2 \times \theta_{x1}$$
$$+ \psi(m_{\text{top of yield line}} + m_{\text{bottom of yield line}}) / 2 \times (\text{vertical length of yield line}) \times \theta_y$$

$$= [\ \rho_b g þ \times \mathbf{AB} \times \mathbf{BF} / 2 + \gamma g \zeta W / W\ ] \times þ C_f/2 \times \theta_{x1}$$
$$+ \psi(m_{\mathbf{AD}} + m_{\mathbf{FG}}) / 2 \times \eta W(1-\varphi) \times \eta(1-\varphi)\theta_{x1} / \zeta$$

$$= [\ \rho_b g þ \times W^2 \zeta \eta(1-\varphi) / 2 + g\gamma\zeta\ ] \times þ C_f/2 \times \theta_{x1}$$
$$+ \eta^2 \theta_{x1}(1-\varphi)^2 \psi / (2\zeta) \times [\ \rho_b g þ \eta W^2(1-\varphi) + 2\gamma g\ ] \times þ C_f / 2$$

AI

$$\psi m_{y=\eta W/2} \times \mathbf{AI} \times \theta_y$$

$$= \psi(m_{\text{top of yield line}} + m_{\text{bottom of yield line}})/2 \times (\text{vertical length of yield line}) \times \theta_y$$

$$= \psi(m_{\mathbf{AD}} + m_{\mathbf{IL}}) / 2 \times \eta W \times \eta(1-\varphi)\theta_{x1} / \zeta$$

$$= \eta^2 \theta_{x1}(1-\varphi)\psi / (2\zeta) \times [\ \rho_b g þ \eta W^2 + 2\gamma g\ ] \times þ C_f / 2$$

Where:

$$m_{\mathbf{AD}} = [\ \rho_b g þ \times 0 + \gamma g W / W\ ] \times þ C_f / 2 / \mathbf{AD}$$
$$= \gamma g þ C_f / (2W)$$

$$m_{\mathbf{FG}} = [\ \rho_b g þ \times \mathbf{BF} \times \mathbf{BC} + \gamma g W(1 - 2\zeta) / W\ ] \times þ C_f / 2 / \mathbf{FG}$$
$$= [\ \rho_b g þ \times \eta W(1-\varphi) \times W(1 - 2\zeta) + \gamma g(1 - 2\zeta)\ ] \times þ C_f / \ [\ (2W(1 - 2\zeta)\ ]$$
$$= [\ \rho_b g þ \times \eta W^2(1-\varphi) + \gamma g\ ] \times þ C_f / (2W)$$

$$m_{\mathbf{IL}} = [\ \rho_b g þ \times \mathbf{AI} \times \mathbf{IL} + \gamma g W/W\ ] \times þ C_f / 2 / \mathbf{IL}$$
$$= [\ \rho_b g þ \times \eta W^2 + \gamma g\ ] \times þ C_f / (2W)$$

9.2.5 The Work Equation's Right-Hand Side

For the right-hand side of the work equation, the total work done in moving a slab is:

$$\int \Delta P \times \text{deflection}\ dA = \iint \Delta P \times \text{deflection}\ dx\ dy = \iint \Delta P \times \text{deflection}\ dy\ dx$$

where both ΔP and deflection are functions of x and y and deflection occurs perpendicular to the x-y plane. Each slab is broken down into subcomponents for the analysis.

Pressure is calculated by $\Delta P = \rho_w g(f_{\text{diff}} - y) + \frac{1}{2}\rho_w v^2 = \Delta P_{y=0} - \rho_w g y$

The deflection equations are:

slab **FGLI**: deflection = $y\delta/(W\eta\varphi)$

slab **AFI**: deflection = $x\delta/(W\zeta)$

slab **DGL**: deflection = $-x\delta/(W\zeta) + \delta/\zeta$

slab **ADGF**: deflection = $-y\delta/(W\eta(1-\varphi)) + \delta/(1-\varphi)$

δ may then be substituted as a function of θ_{x1}.

For $W\eta > f_{diff} > W\eta\varphi$, the calculations are illustrated for **ABF**, **AEF**, and **FGJK**:

ABF for $W\eta > f_{diff} > W\eta\varphi$

$$\int_{\mathbf{EF}}^{y = f_{diff}} \int_{\mathbf{AF}}^{\mathbf{BF}} \Delta P \times \text{deflection } dx\, dy$$

$$= \int_{y = W\eta\varphi}^{y = f_{diff}} \int_{x = \zeta(W\eta - y)/[\eta(1-\varphi)]}^{x = W\zeta} [\, \Delta P_{y=0} - \rho_w g y \,] \times [\, -y\delta/(W\eta(1-\varphi)) + \delta/(1-\varphi) \,]\; dx\, dy$$

$$= \theta_{x1}\zeta/(\eta(1-\varphi)) \times [\, \Delta P_{y=0} \times (f_{diff}^2 W\eta(1+\varphi)/2 - f_{diff}^3/3 - f_{diff} W^2\eta^2\varphi + W^3\eta^3\varphi^2/2 - W^3\eta^3\varphi^3/6)$$
$$+ g\rho_w \times (f_{diff}^4/4 - f_{diff}^3 W\eta(1+\varphi)/3 + f_{diff}^2 W^2\eta^2\varphi/2 + W^4\eta^4\varphi^4/12 - W^4\eta^4\varphi^3/6) \,]$$

AEF for $W\eta > f_{diff} > W\eta\varphi$

$$\int_{\mathbf{EF}}^{y = f_{diff}} \int_{\mathbf{AE}}^{\mathbf{AF}} \Delta P \times \text{deflection } dx\, dy$$

$$= \int_{y = W\eta\varphi}^{y = f_{diff}} \int_{x = 0}^{x = \zeta(W\eta - y)/[\eta(1-\varphi)]} [\, \Delta P_{y=0} - \rho_w g y \,] \times [\, x\delta/(W\zeta) \,]\; dx\, dy$$

$$= \theta_{x1}\zeta/(2\eta(1-\varphi)) \times [\, \Delta P_{y=0} \times (f_{diff} W^2\eta^2 + f_{diff}^3/3 - f_{diff}^2 W\eta - W^3\eta^3\varphi - W^3\eta^3\varphi^3/3 + W^3\eta^3\varphi^2)$$
$$+ g\rho_w \times (2f_{diff}^3 W\eta/3 - f_{diff}^4/4 - f_{diff}^2 W^2\eta^2/2 - 2W^4\eta^4\varphi^3/3 + W^4\eta^4\varphi^4/4 + W^4\eta^4\varphi^2/2) \,]$$

FGKJ for $W\eta > f_{diff} > W\eta\varphi$

$$\int_{\mathbf{JK}}^{\mathbf{FG}} \int_{\mathbf{FJ}}^{\mathbf{GK}} \Delta P \times \text{deflection } dx\, dy$$

$$= \int_{y=0}^{y=W\eta\varphi} \int_{x=W\zeta}^{x=W(1-\zeta)} [\, \Delta P_{y=0} - \rho_w g y \,] \times [\, y\delta / (W\eta\varphi) \,]\; dx\, dy$$

$$= \theta_{x1} W^3 (1-2\zeta)\eta^2\varphi(1-\varphi) \times [\, \Delta P_{y=0}/2 - gW\eta\rho_w\varphi/3 \,]$$

For $0 < f_{diff} < W\eta\varphi$, **ABF**, **AEF**, **BCGF**, **CDG**, and **DGH** will contribute zero work done because pressure is not imposed on these slabs. Calculations are illustrated for **EFI**, **FGJK**, and **FIJ**:

EFI for $0 < f_{diff} < W\eta\varphi$

$$\int_{\mathbf{IJ}}^{y=f_{diff}} \int_{\mathbf{EI}}^{\mathbf{FI}} \Delta P \times \text{deflection } dx\, dy$$

$$= \int_{y=0}^{y=f_{diff}} \int_{x=0}^{x=y\zeta/(\eta\varphi)} [\, \Delta P_{y=0} - \rho_w g y \,] \times [x\delta/(W\zeta)\,]\; dx\, dy$$

$$= \theta_{x1}\, f_{diff}^3 \zeta(1-\varphi)/(2\eta\varphi^2) \times [\, \Delta P_{y=0}/3 - f_{diff}\, g\rho_w/4 \,]$$

FGKJ for $0 < f_{diff} < W\eta\varphi$

$$\int_{\mathbf{JK}}^{y=f_{diff}} \int_{\mathbf{FJ}}^{\mathbf{GK}} \Delta P \times \text{deflection } dx\, dy$$

$$= \int_{y=0}^{y=f_{diff}} \int_{x=W\zeta}^{x=W(1-\zeta)} [\, \Delta P_{y=0} - \rho_w g y \,] \times [\, y\delta / (W\eta\varphi) \,]\; dx\, dy$$

$$= \theta_{x1} f_{diff}^2 W(1-2\zeta)(1-\varphi)/\varphi \times [\, \Delta P_{y=0}/2 - f_{diff}\, g\rho_w/3 \,]$$

FIJ for $0 < f_{diff} < W\eta\varphi$

$$\int_{\mathbf{IJ}}^{y = f_{diff}} \int_{\mathbf{FI}}^{\mathbf{FJ}} \Delta P \times \text{deflection}\ dx\ dy$$

$$= \int_{y = 0}^{y = f_{diff}} \int_{x = y\zeta/(\eta\varphi)}^{x = W\zeta} [\ \Delta P_{y=0} - \rho_w g y\ ] \times [\ y\delta / (W\eta\varphi)\ ]\ dx\ dy$$

$$= \theta_{x1} f_{diff}^2 \zeta(1-\varphi)/\varphi \times [\ \Delta P_{y=0} \times (W/2 - f_{diff}/(3\eta\varphi)) + g\rho_w \times (f_{diff}^2/(4\eta\varphi) - f_{diff}W/3)\ ]$$

For $f_{diff} > W\eta$, the calculations follow a similar pattern. Only **CDG** is illustrated:

CDG for $f_{diff} > W\eta$

$$\int_{\mathbf{GH}}^{\mathbf{CD}} \int_{\mathbf{CG}}^{\mathbf{DG}} \Delta P \times \text{deflection}\ dx\ dy$$

$$= \int_{y = W\eta\varphi}^{y = W\eta} \int_{x = W(1-\zeta)}^{x = y\zeta/[\eta(1-\varphi)] - W\zeta/(1-\varphi) + W} [\ \Delta P_{y=0} - \rho_w g y\ ] \times [\ -y\delta/(W\eta(1-\varphi)) + \delta/(1-\varphi)\ ]\ dx\ dy$$

$$= \theta_{x1}\zeta/(1-\varphi) \times [\ \Delta P_{y=0} W^3\eta^2 \times (-\varphi^3/6 + \varphi^2/2 - \varphi/2 + 1/6)$$
$$+ g\rho_w W^4\eta^3 \times (\varphi^4/12 - \varphi^3/6 + \varphi/6 - 1/12)\ ]$$

9.2.6 Solving the Work Equation

Equating the right-hand and left-hand sides yields m as a function of f_{diff} and v (or of $\Delta P_{y=0}$ and f_{diff}) for each slab. For minimum collapse pressure, ΔP is differentiated with respect to ζ and φ and these equations are set to 0:

$$\partial \Delta P / \partial \zeta = 0 \qquad \partial \Delta P / \partial \varphi = 0$$

This system is two equations in two unknowns which is solvable. Solving for ζ and φ provides ΔP_{min}. The f_{diff} terms complicate the differentiation and solving numerically is most appropriate.

9.3 Parameters

9.3.1 Selecting Parameters

To find ΔP_{min}, the parameters described in this Section are needed. Each parameter is presented with the value chosen followed by the sources which contributed to the selection of that value. The decisions and parameter names are summarised in Section 9.3.2. The sensitivity of the results to these parameter choices is examined in Section 9.4.3.

$A \in \{38\ m^2, 55\ m^2, 84\ m^2\}$ (Chapter 6).

b (height of ground floor above ground level) = 0 to be generic.

Results presented for f_{diff} must have b added just before interpretation in order to present the true situation.

$C_f = 0.95$ (Spence, 2001-2).

$g = 9.80665\ m/s^2$ (Lide, 2001).

H = 2.4 m (Chapter 6).

H in this instance refers to the distance from the bottom to the top of the ground floor. For cavity wall failure, this distance strictly refers to only the blockwork inner leaf which has a continuous support where it meets the floor at the bottom of the ground floor. The brickwork outer leaf extends down to the ground (Figures 9.1.4.1, 9.1.4.2, and 9.2.1.2) so that H for the outer leaf is normally 0.2-0.3 m greater than H for the inner leaf. If Southcombe and Tapp (1988) are correct that damp proof courses provide a nominal degree of fixity (Section 9.1.2), then this difference in H would be slightly less. Furthermore, the leaves of some cavity walls are physically attached by connecting air bricks below floor level (Chudley, 1995) which would also add a degree of fixity to the outer leaf, reducing its effective H. Finally, the same argument from Figure 9.1.4.3 applies to this case. Therefore, for these calculations, H = 2.4 m will be used, but the sensitivities of the results to a 0.3 m variation in H will be examined in Section 9.4.3.

$j \in \{1, 2, 3, 4\}$ (Chapter 6).

$W \in \{1.0\text{ m}, 2.0\text{ m}, 3.0\text{ m}, 4.0\text{ m}, 5.0\text{ m}, 6.0\text{ m}\}$.

The residence survey (Chapter 5) could not examine internal characteristics, thus these panel widths were inferred from house plans obtained from estate agents in the case study sites and books on architectural design of residences in the case study areas (e.g. ASPP, 1972; Godwin, c. 1955; House Plans, 1975; Marley, 1954). W = 1.0 m and W = 6.0 m are relatively rare but represent extremes.

To calculate γ, three parameters are needed.

First, A which has already been provided.

Second, the loading from upper storeys, which varies depending on j and the panel's location within the residence, is needed. For conservative estimates of failure ΔP, an empty residence is assumed, since people may have evacuated and removed possessions due to flood warnings. Therefore, only the dead load—the mass of the structure and fixed parts (the permanent mass) of the residence including walls, floors, ceilings, and cladding—of the residence is considered for a conservative estimate.

For dead load, the mass of the blockwork above the inner leaf of the ground floor panel is:

$$\rho_{blockwork} HW\beta_{blockwork} \times (j - 1)$$

The blockwork inner leaf also takes the rest of the residence's dead load which is predominantly floors ($\approx$50 kg/m^2 per storey) and the roof ($\approx$70 kg/m^2) (McEvoy, 2001-2; Spence 2001-2). Each panel will take some fraction, λ, of this dead load. Thus, the total mass on the inner leaf is:

$$\gamma_{blockwork} = \rho_{blockwork} HW\beta_{blockwork} \times (j - 1) + [\ (\text{floor kg/m}^2) \times (j - 1) + (\text{roof kg/m}^2)\ ] \times A\lambda_{blockwork}$$

For the brickwork outer leaf, the dead load has the same formula as for blockwork except that $\lambda_{brickwork} = 0$. The mass results from only the brickwork above the panel:

$$\gamma_{brickwork} = \rho_{brickwork} HW\beta_{brickwork} \times (j - 1)$$

Third, $\lambda_{blockwork}$ is needed. This factor accounts for distribution of the non-wall dead load amongst the residence's wall panels, both internal and external. For an external wall, $\lambda_{blockwork}$ = 0.2 is reasonable because the realistic minimum is 0 and a realistic maximum is approximately $^1/_3$, although the absolute maximum would be ½ if the residence's entire load is

distributed over two opposite external walls (McEvoy, 2001-2). The panel which fails, though, is not necessarily the entire external wall because internal supports subdivide external walls into various panels. Since H does not vary, the ratio of panel width to wall width is needed. This value varies between approximately 0.1 and exactly one. Thus, $\lambda_{blockwork}$, the overall factor to account for load distribution amongst external panels, is realistically considered to be between 0.0 and 0.4. For minimum dead load $\lambda_{blockwork}$ is taken as 0.0, for average dead load $\lambda_{blockwork}$ is taken as 0.2, and for maximum dead load $\lambda_{blockwork}$ is taken as 0.4.

$\rho_{brickwork} = 1{,}900$ kg/m^3

A value just under 2,000 kg/m^3 was deemed appropriate (see Table 9.3.1.1).

Table 9.3.1.1: Density of Brickwork

Source	Value or Comments
Adler (1999)	1,920 kg/m^3
Brick Development Association (c. 2002)	21.6 kN/m^3
Ibstock Brick (2001)	1,400 to 2,000 kg/m^3
Onesteel (2000)	1,950 kg/m^3

$\rho_{blockwork} = 1{,}000$ kg/m^3

Few sources provide the density of blockwork specifically used in residences, so an estimate taking into account the best data available (see Table 9.3.1.2) was made.

Table 9.3.1.2: Density of Blockwork

Source	Value or Comments
Adler (1999)	concrete's density ≈ 500-2000 kg/m^3
Foster (2000)	Inner leaf concrete blocks are lightweight.
Blyth (1989)	"Blocks which have a block density of not less than 1500 kg/m^3 are suitable for general use in buildings".

$\rho_w = 1{,}026$ kg/m^3 (Lide, 2001)

$\psi = 0.3$

This value is from Spence (2001-2) providing a conservative estimate. Martini (1998) uses $\psi = 1.0$ without reference or comment except to note that "this value has not been verified by experiment". Martini (2002) added that he had done a recent literature search and had not found studies that had researched the value of ψ through experiments. Khanna (1996) suggests that "The coefficient of friction is the tangent of the angle of internal friction" and provides approximate values for some materials. For example, "Masonry on same or brick" is 0.70, masonry on dry clay is 0.50, and "Cement blocks on same" is 0.65 (Khanna, 1996). No values are given for masonry units on mortar. Keeping ψ relatively low at 0.3 is therefore conservative.

$þ_{brickwork} = 0.1025$ m (Adler, 1999; BS 3921 : 1985).

$þ_{blockwork} = 0.1000$ m

Few sources provide the thickness of blockwork specifically used in residences, so an estimate taking into account the data available (see Table 9.3.1.3) is made.

Table 9.3.1.3: Thickness of Blockwork

Source	Value or Comments
Blyth (1989)	If loadbearing, greater than 0.075 m and probably around 0.100 m.
BS 5628-3 : 2001	"Cavity walls consist of: a) an outer leaf, 100 mm minimum in thickness, of any brick, or blocks of compressive strength not less than 14.0 N/mm^2; b) an inner leaf, 100 mm minimum in thickness, of any brick, or blocks of compressive strength not less than 3.5 N/mm^2."
BS 6073-2 : 1981	Standard sizes provided are from 0.060 to 0.250 m at 10-15 mm intervals.
Foster (2000)	Inner leaves are "more commonly" made of "lightweight concrete blocks 100 mm thick" even when the outer leaf is similar.
Powell-Smith and Billington (1990)	The sum of the thicknesses of the two leaves should be greater than 0.200 mm according to the building regulations.

9.3.2 Summary of Parameters

Constants: b (height of ground floor above ground level) = 0 to be generic, but results presented for f_{diff} must have b added just before interpretation in order to present the true situation.

C_f (compression depth factor) = 0.95

g (gravitational acceleration) = 9.80665 m/s^2

H (panel height) = 2.4 m

$\rho_{blockwork}$ (blockwork density) = 1,000 kg/m^3

$\rho_{brickwork}$ (brickwork density) = 1,900 kg/m^3

ρ_w (water density) = 1,026 kg/m^3

ψ (friction coefficient) = 0.3

$þ_{blockwork}$ (blockwork thickness) = 0.1000 m

$þ_{brickwork}$ (brickwork thickness) = 0.1025 m

The inner leaf blockwork takes the residence's entire non-wall dead load which is floors (50 kg/m^2 per storey) and the roof (70 kg/m^2).

Variables: A (residence plan area) $\in$ {38 m^2, 55 m^2, 84 m^2}

j (# storeys) $\in$ {1, 2, 3, 4}

W (panel width) $\in$ {1.0 m, 2.0 m, 3.0 m, 4.0 m, 5.0 m, 6.0 m}

λ (the load factor which accounts for distribution of the non-wall dead load amongst the residence's wall panels, both internal and external).

- $\lambda_{brickwork}$ = 0.0 because the inner leaf takes all the residence's load, hence the only load on brickwork is from the brickwork above it.
- $\lambda_{blockwork}$: $f_{diff\,min}$ for failure is calculated with $\lambda_{blockwork}$ = 0.0
  $f_{diff\,avg}$ for failure is calculated with $\lambda_{blockwork}$ = 0.2
  $f_{diff\,max}$ for failure is calculated with $\lambda_{blockwork}$ = 0.4
- Imposed/live loads are assumed to be zero.

Parameters Determined: f_{diff} (flood depth differential) in m

v (flood velocity) in m/s

9.4 Results

9.4.1 f_{diff} for v = 0

Figures 9.4.1.1, 9.4.1.2, 9.4.1.3, and 9.4.1.4 depict f_{diff} at which a wall panel would fail as a function of W assuming that v = 0 m/s. These failure f_{diff} values are not directly translatable into failure ΔP because the failure ΔP_{min} changes as f_{diff} changes. Neither $f_{diff\ min}$ for blockwork nor f_{diff} for brickwork change with A because $\lambda = 0.0$.

Figure 9.4.1.1: Failure f_{diff} vs W for v = 0 m/s and j = 1
(Curves are for blockwork unless indicated as brickwork.)

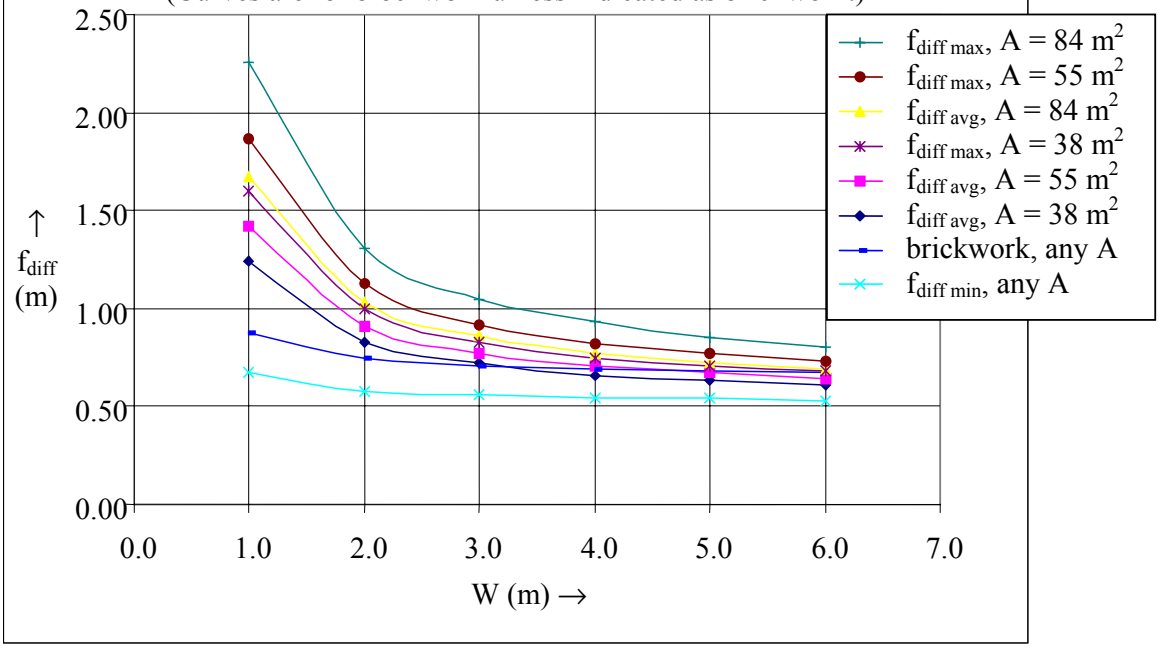


Figure 9.4.1.2: Failure f_{diff} vs W for v = 0 m/s and j = 2
(Curves are for blockwork unless indicated as brickwork.)

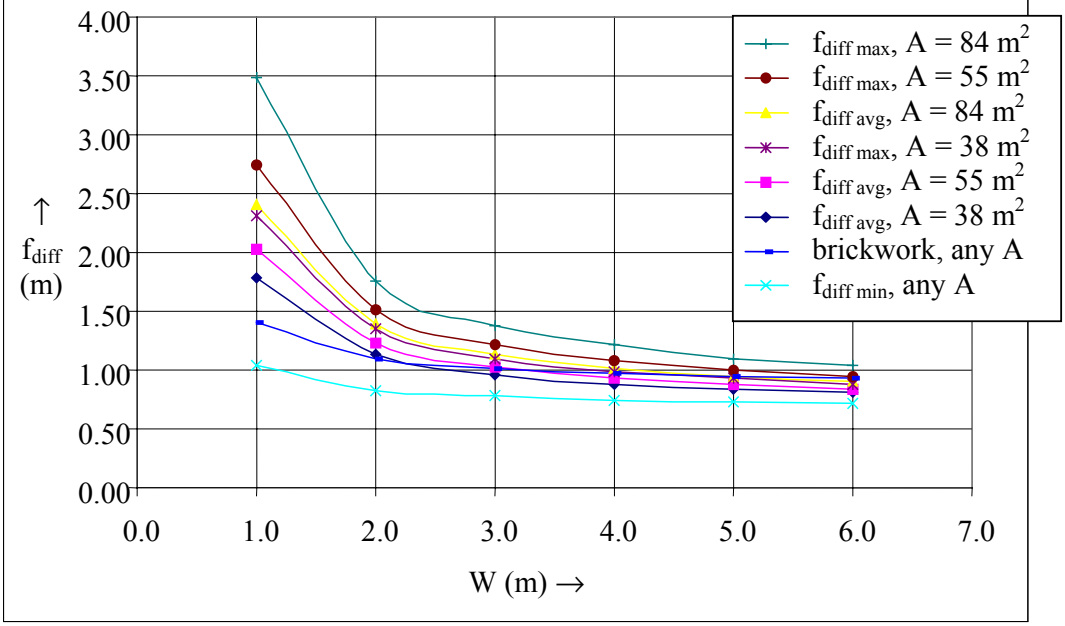


Figure 9.4.1.3: Failure f_{diff} vs W for v = 0 m/s and j = 3
(Curves are for blockwork unless indicated as brickwork.)

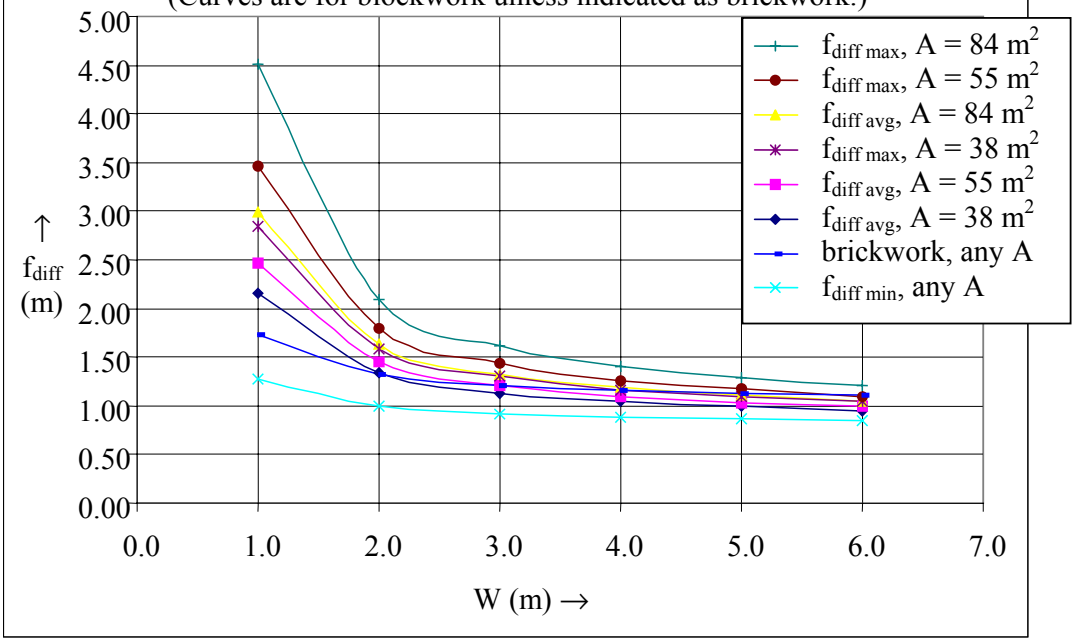
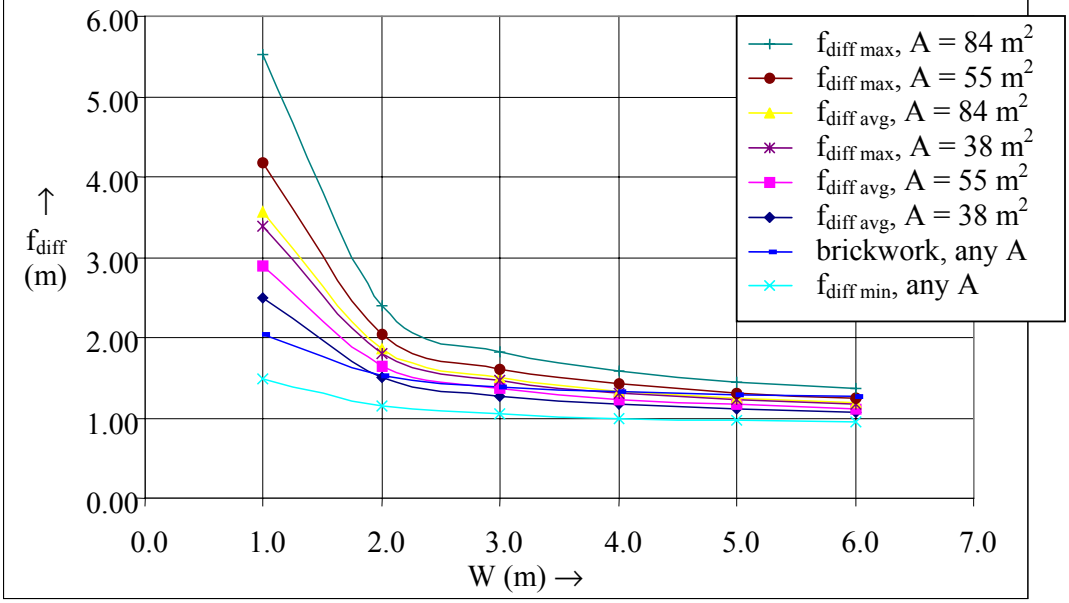


Figure 9.4.1.4: Failure f_{diff} vs W for v = 0 m/s and j = 4
(Curves are for blockwork unless indicated as brickwork.)



9.4.2 f_{diff}-v Failure Curves

In Figures 9.4.2.1 through 9.4.2.8, one curve has been produced for each A-j-W-λ combination where A ∈ (38 m^2, 55 m^2, 84 m^2), j ∈ (1,2,3,4), W ∈ (2.0 m, 5.0 m) representing narrow and wide wall panels respectively, and λ ∈ (0.0,0.4) representing minimum and maximum possible failure scenarios respectively.

The region to the right (and above) each curve is the zone of failure whereas the region to the left (and below) each curve is the zone of non-failure. The curves for a specific W may be paired according to high $\lambda_{blockwork}$ = 0.4 and low $\lambda_{blockwork}$ = 0.0. Only one curve for $\lambda_{blockwork}$ = 0.0 is provided for all A because the curves are the same for any A. The region in between the two curves of $\lambda_{blockwork}$ = 0.0 and $\lambda_{blockwork}$ = 0.4 is where failure may or may not occur, depending on how much dead load the blockwork panel supports. Brickwork is provided in each graph for reference.

Figure 9.4.2.1: f_{diff}-v Failure Curves
for j = 1 and Narrow Wall Panels (W = 2.0 m)

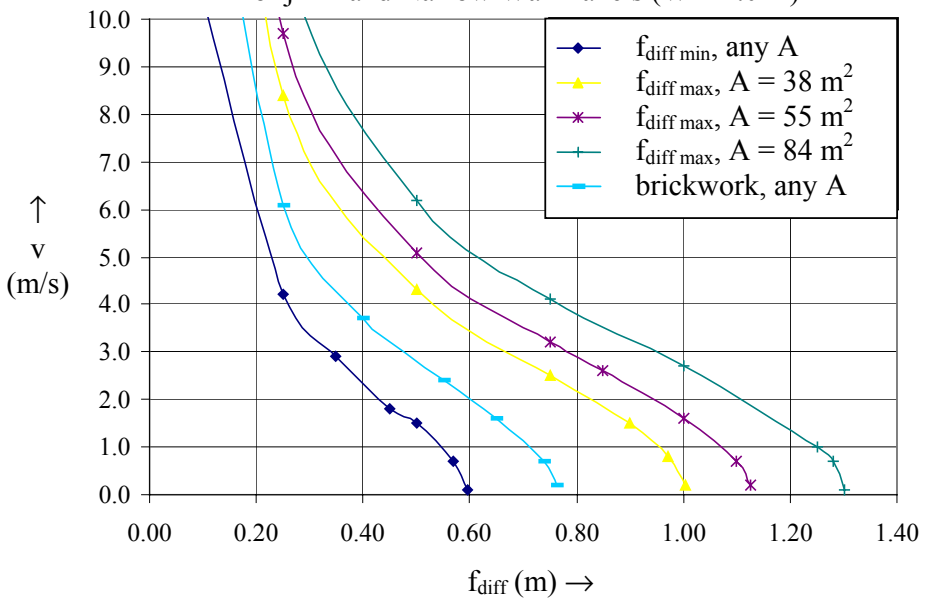


Figure 9.4.2.2: f_{diff}-v Failure Curves
for j = 1 and Wide Wall Panels (W = 5.0 m)

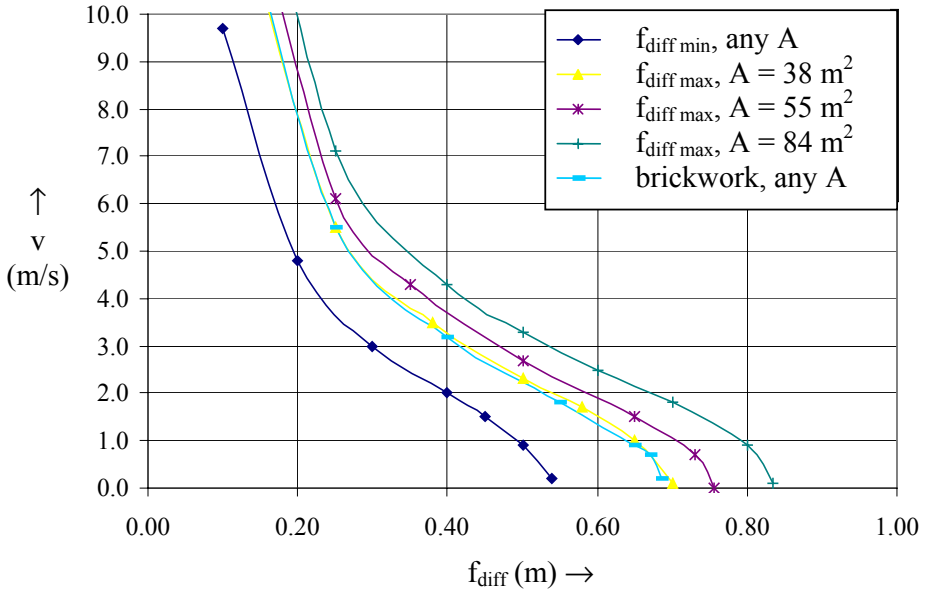


Figure 9.4.2.3: f_{diff}-v Failure Curves
for j = 2 and Narrow Wall Panels (W = 2.0 m)

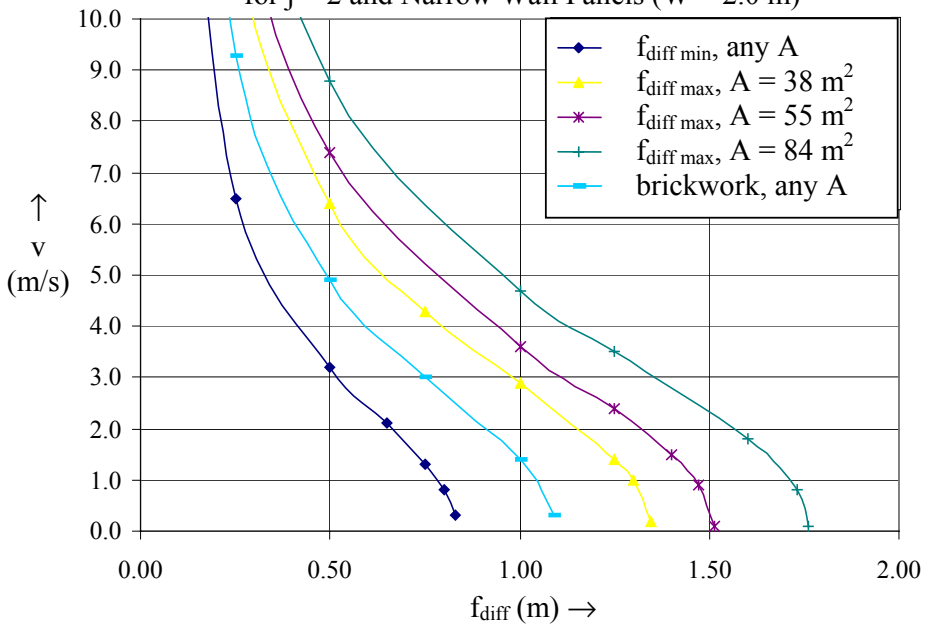


Figure 9.4.2.4: f_{diff}-v Failure Curves
for j = 2 and Wide Wall Panels (W = 5.0 m)

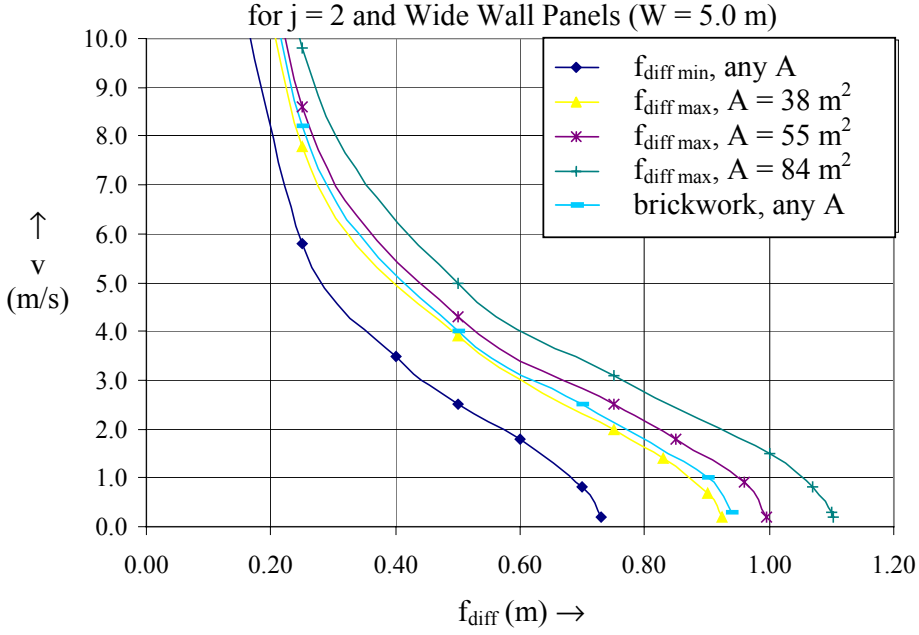


Figure 9.4.2.5: f_{diff}-v Failure Curves
for j = 3 and Narrow Wall Panels (W = 2.0 m)

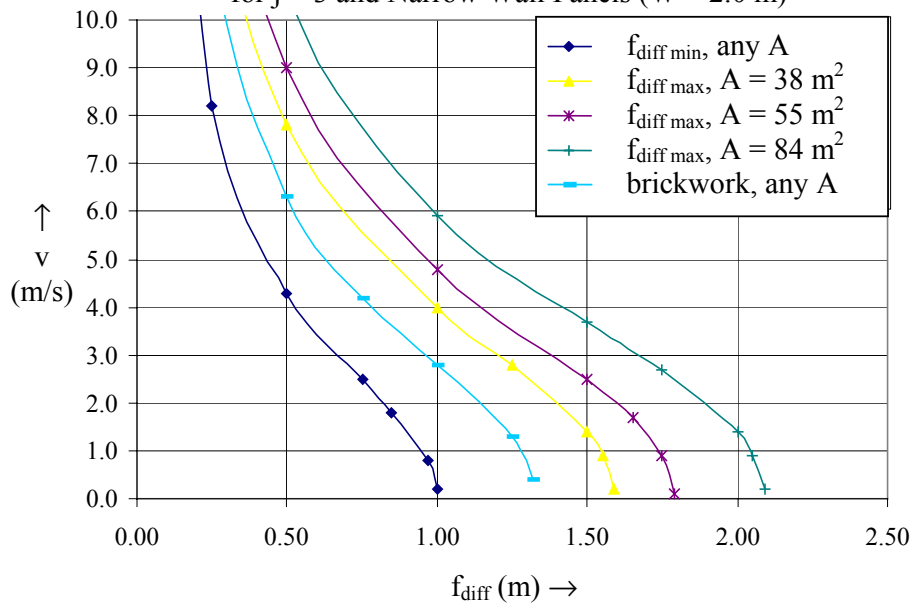


Figure 9.4.2.6: f_{diff}-v Failure Curves
for j = 3 and Wide Wall Panels (W = 5.0 m)

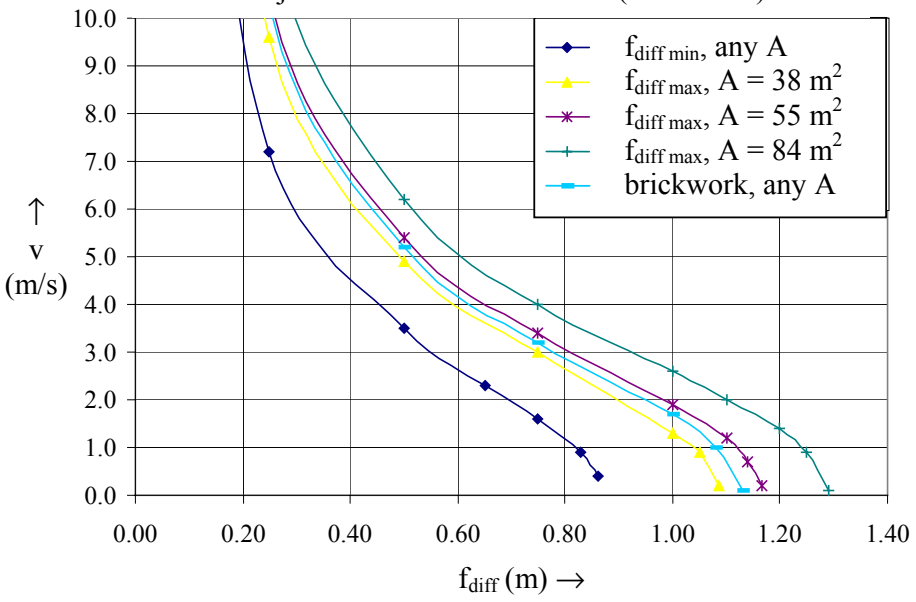


Figure 9.4.2.7: f_{diff}-v Failure Curves
for j = 4 and Narrow Wall Panels (W = 2.0 m)

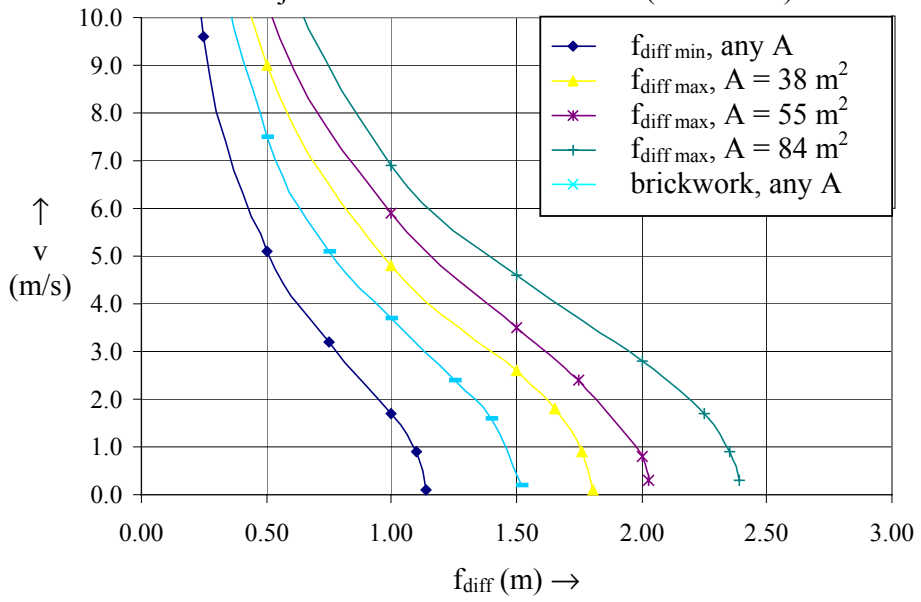
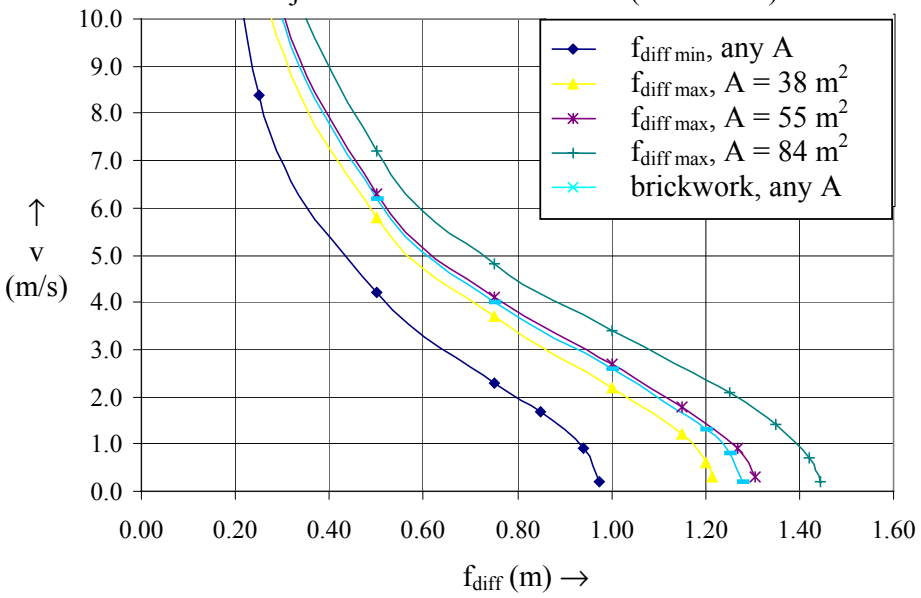


Figure 9.4.2.8: f_{diff}-v Failure Curves
for j = 4 and Wide Wall Panels (W = 5.0 m)



9.4.3 Sensitivity Analysis

To determine the influence on the results of the parameters deemed to be fixed, variations were examined by taking for comparison a typical wall with the following values:

$A = 55\ m^2$

$j = 2$

$W = 2.0\ m, 5.0\ m$

$\lambda_{brickwork} = 0.0$

$\lambda_{blockwork}$: $f_{diff\ min}$ for failure is calculated with $\lambda_{blockwork} = 0.0$

$f_{diff\ max}$ for failure is calculated with $\lambda_{blockwork} = 0.4$

Variations: $C_f \in \{0.85, 0.95, 1.00\}$

$H \in \{2.1\ m, 2.4\ m, 2.7\ m\}$

$\rho_{blockwork} \in \{500\ kg/m^3, 1{,}000\ kg/m^3, 2{,}000\ kg/m^3\}$

$\psi \in \{0.1, 0.3, 0.5\}$

$\beta_{blockwork} \in \{0.1000, 0.1500\ m\}$

A variation of ± 25% was taken for each of the floor and roof dead loads.

For Table 9.4.3.1, one variable at a time is varied with all others being held at their original levels, except for the two dead loads which were varied +25% simultaneously and –25% simultaneously. All calculations in Table 9.4.3.1 are completed for blockwork. Varying $\rho_{blockwork}$ with $\lambda = 0.0$ effectively compares brickwork and blockwork since $\beta_{blockwork} \approx \beta_{brickwork}$.

Table 9.4.3.1: f_{diff} for v = 0 m/s Varying Parameters Individually

Variable	(W,λ) = (2.0 m, 0.0)	(W,λ) = (5.0 m, 0.0)	(W,λ) = (2.0 m, 0.4)	(W,λ) = (5.0 m, 0.4)
$C_f = 0.85$	0.80 m	0.70 m	1.44 m	0.96 m
$C_f = 0.95$	0.83 m	0.73 m	1.51 m	1.00 m
$C_f = 1.00$	0.85 m	0.74 m	1.54 m	1.02 m
H = 2.1 m	0.79 m	0.70 m	1.50 m	0.99 m
H = 2.4 m	0.83 m	0.73 m	1.51 m	1.00 m
H = 2.7 m	0.88 m	0.76 m	1.54 m	1.01 m
$\rho_{blockwork} = 500$ kg/m^3	0.64 m	0.57 m	1.44 m	0.91 m
$\rho_{blockwork} = 1{,}000$ kg/m^3	0.83 m	0.73 m	1.51 m	1.00 m
$\rho_{blockwork} = 1{,}500$ kg/m^3	0.98 m	0.85 m	1.59 m	1.08 m
$\rho_{blockwork} = 2{,}000$ kg/m^3	1.09 m	0.95 m	1.65 m	1.15 m
$\psi = 0.1$	0.76 m	0.71 m	1.34 m	0.96 m
$\psi = 0.3$	0.83 m	0.73 m	1.51 m	1.00 m
$\psi = 0.5$	0.89 m	0.75 m	1.65 m	1.03 m
$\beta_{blockwork} = 0.1000$ m	0.83 m	0.73 m	1.51 m	1.00 m
$\beta_{blockwork} = 0.1500$ m	1.15 m	0.99 m	1.90 m	1.26 m
floor dead load is 25 kg/m^2 roof dead load is 35 kg/m^2	0.83 m	0.73 m	1.23 m	0.88 m
floor dead load is 35 kg/m^2 roof dead load is 50 kg/m^2	0.83 m	0.73 m	1.36 m	0.93 m
floor dead load is 50 kg/m^2 roof dead load is 70 kg/m^2	0.83 m	0.73 m	1.51 m	1.00 m
floor dead load is 65 kg/m^2 roof dead load is 90 kg/m^2	0.83 m	0.73 m	1.65 m	1.06 m
floor dead load is 100 kg/m^2 roof dead load is 140 kg/m^2	0.83 m	0.73 m	1.96 m	1.19 m

Table 9.4.3.1 illustrates that varying C_f and H produces negligible differences. Therefore, any potential difference in H between the blockwork and brickwork leaves (up to 0.3 m, the maximum generally expected) has negligible effect on the results. Varying ψ and the dead loads yields a potentially significant difference, depending on other parameters. This difference is not critical because even wide variations could lead to a maximum difference in f_{diff} of 0.5 m (one column across in the vulnerability matrices; see Chapter 12). Nonetheless, further examination is warranted. Table 9.4.3.2 considers the four parameters discussed so far in the extreme cases of each parameter at its maximum value or each parameter at its minimum value.

Table 9.4.3.2: f_{diff} for v = 0 m/s Varying C_f, H, ψ, and the Dead Loads Simultaneously

Variable	(W,λ) = (2.0 m, 0.0)	(W,λ) = (5.0 m, 0.0)	(W,λ) = (2.0 m, 0.4)	(W,λ) = (5.0 m, 0.4)
C_f = 0.85 H = 2.1 m ψ = 0.1 floor dead load is 35 kg/m^2 roof dead load is 50 kg/m^2	0.69 m	0.65 m	1.15 m	0.84 m
C_f = 1.00 H = 2.7 m ψ = 0.5 floor dead load is 65 kg/m^2 roof dead load is 90 kg/m^2	0.96 m	0.80	1.88 m	1.13 m

The only case for concern is (W,λ) = (2.0 m, 0.4) which displays a large difference. Since all parameters are unlikely to coincide in this extreme situation, it is reasonably certain that the values chosen for these parameters are acceptable for the desired first-order analysis.

In Table 9.4.3.1, varying $\rho_{blockwork}$ and $þ_{blockwork}$ produces differences significant enough to warrant further examination. These two parameters are likely to be the most controllable and easily changed out of the parameters in Table 9.4.3.1. The two extreme cases are taken for panel properties: (W,λ) = (2.0 m, 0.4) for the strongest panel and (W,λ) = (5.0 m, 0.0) for the weakest panel. f_{diff}-v failure curves appear in Figures 9.4.3.1 through 9.4.2.4.

$\rho_{blockwork}$ changed failure f_{diff} up to 0.5 m, but changing $þ_{blockwork}$ can double failure f_{diff}. While the choices of the original values selected are supported by the literature, altering the blockwork used in residence construction has consequences which are discussed in Section 9.5.

Figure 9.4.3.1: f_{diff}-v Failure Curves for $\beta_{blockwork}$ = 0.075 m

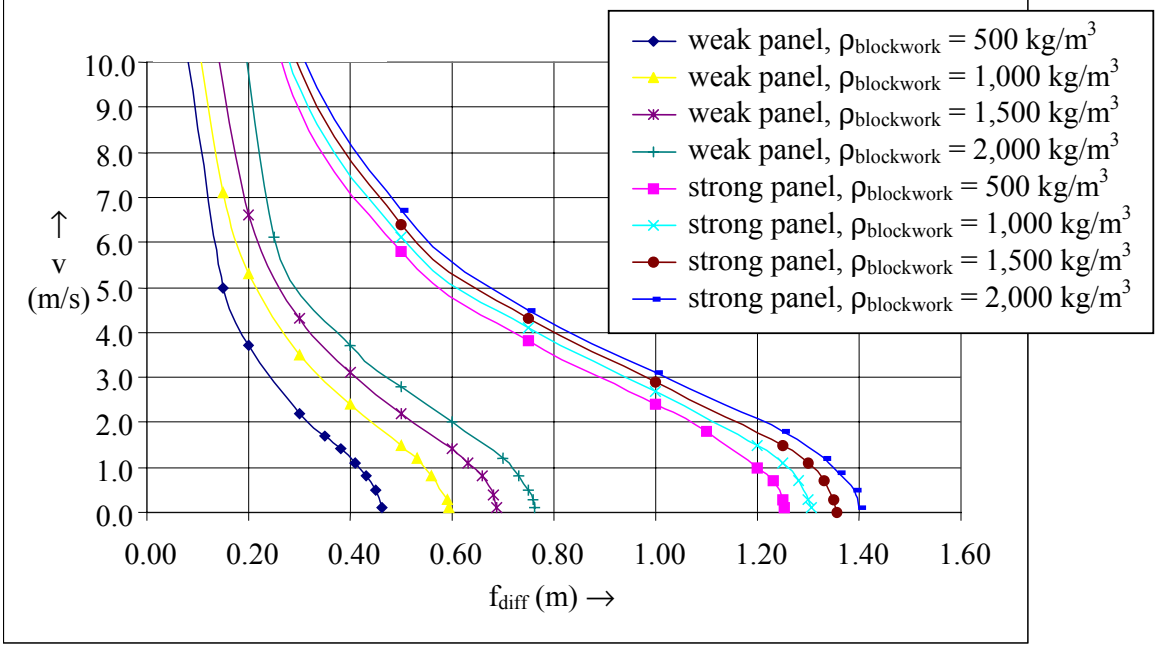


Figure 9.4.3.2: f_{diff}-v Failure Curves for $\beta_{blockwork}$ = 0.200 m

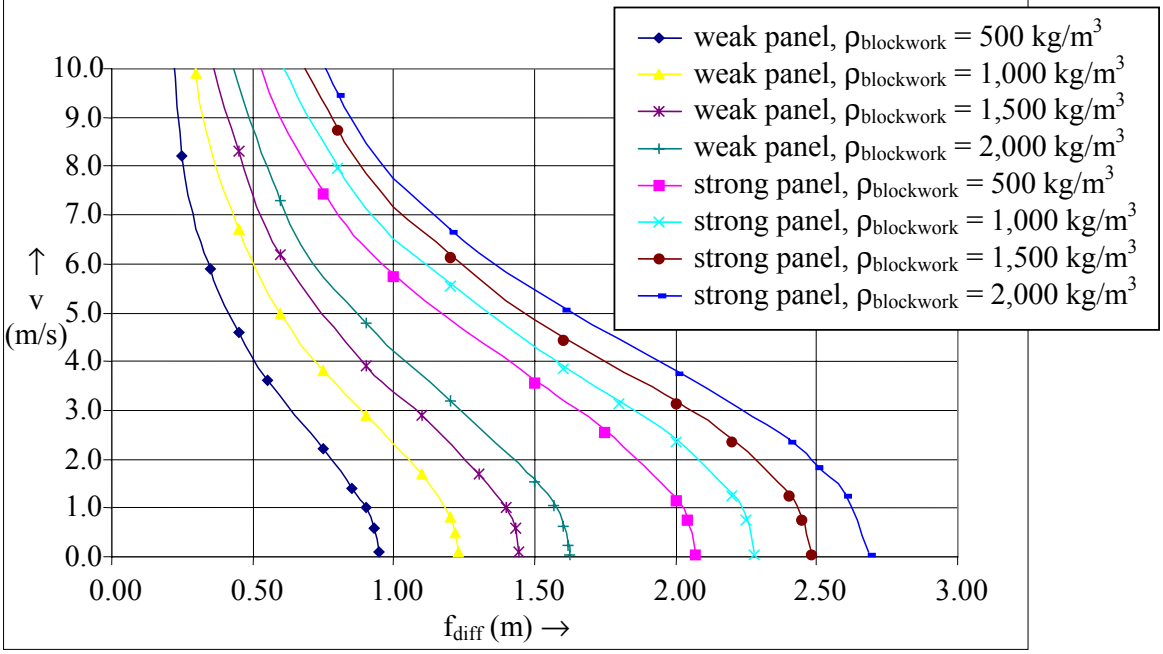


Figure 9.4.3.3: f_{diff}-v Failure Curves for $\rho_{blockwork}$ = 500 kg/m^3

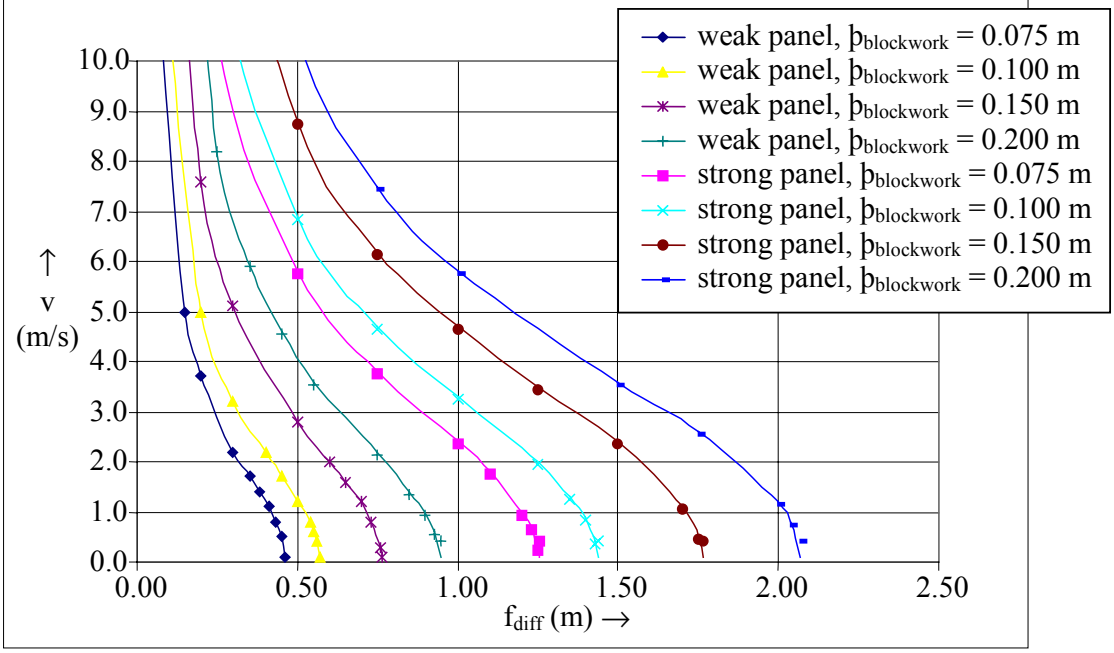
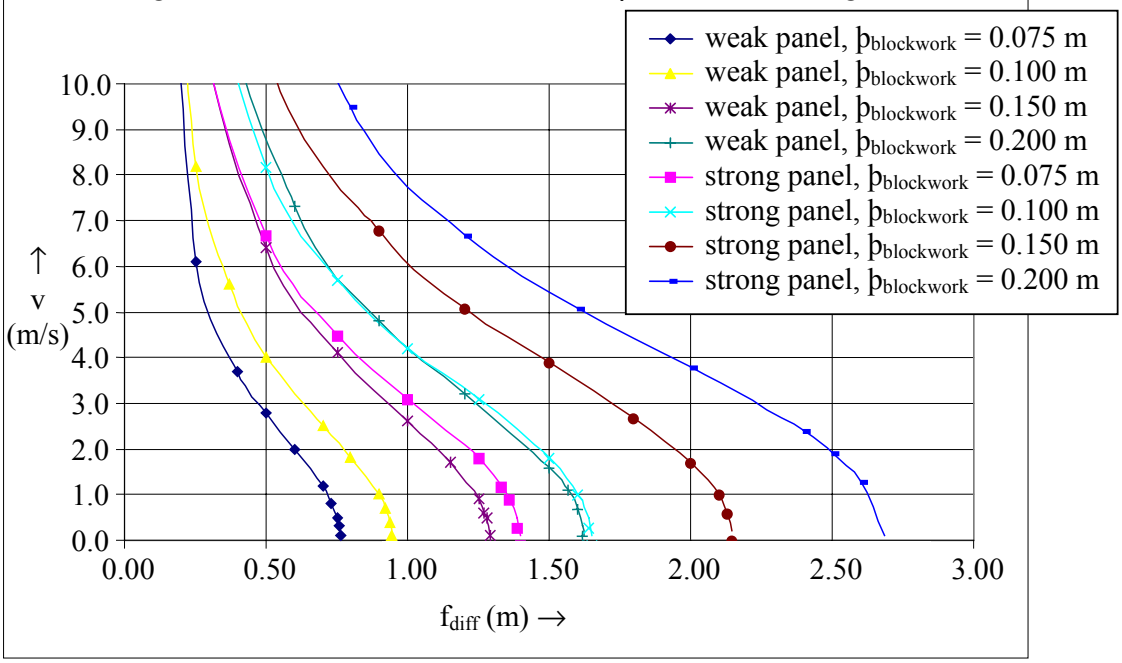


Figure 9.4.3.4: f_{diff}-v Failure Curves for $\rho_{blockwork}$ = 2,000 kg/m^3



9.5 Discussion

9.5.1 Strength of Wall Panels

The range of $\Delta P_{y=0}$ under which panels start to fail when v = 0 m/s runs from 8 kPa for the weakest panel to 55 kPa for the strongest panel with the majority between 10 kPa and 20 kPa. The range of ΔP_{avg} is generally from just over 1 kPa to approximately 40 kPa with the most common being between just over 1 kPa and 10 kPa. This range holds even when v > 0 m/s. Some weak panels have 0.5 kPa < ΔP_{avg} < 1.0 kPa for v < 5 m/s and thus relatively high f_{diff}. Some strong panels have ΔP_{avg} > 10 kPa for the unrealistic situation of v > 30 m/s and f_{diff} < 0.20 m.

This range for ΔP_{avg} conforms with the range found in experimental studies of uniform loading (Section 9.1.2), somewhat validating the method and assumptions chosen. One outcome from these calculations is that defining a specific $\Delta P_{y=0}$ or ΔP_{avg} for a panel's failure is not possible as with uniform loading. The combination of hydrostatic and hydrodynamic pressures means that ΔP_{min} for failure shifts according to the f_{diff}-v combination. Either f_{diff} or v must be defined before ΔP_{min} for failure may be calculated.

Analysing the yield line pattern (Figure 9.2.1.1) indicated that most panels break with H > f_{diff} > φH although the strongest panels break at f_{diff} > H. Panels never break for f_{diff} < φH. This result arises because the ΔP required to break the panel at f_{diff} < φH implies a velocity so high that a lower ΔP_{min} is achieved by a break pattern with f_{diff} > φH.

For v = 0 m/s, wall panels start failing at approximately f_{diff} = 0.8 m. Most wall panels would have failed by f_{diff} = 2.0 m. This 1.2 m range is relatively narrow, indicating a fairly rapid jump across a threshold from non-structural failure, such as glass breaking or infiltration, to structural failure. If a residence were completely sealed and f_{diff} ≈ 1.0 m, the swell from non-breaking waves or from boats or vehicles travelling through the water could raise f_{diff} over the threshold and precipitate structural failure.

From strongest to weakest, the sequence of panels with one exception is always:

1. blockwork $\lambda_{blockwork}$ = 0.4 ($f_{diff\,max}$) A = 84 m^2
2. blockwork $\lambda_{blockwork}$ = 0.4 ($f_{diff\,max}$) A = 55 m^2
3. blockwork $\lambda_{blockwork}$ = 0.2 ($f_{diff\,avg}$) A = 84 m^2

4. blockwork $\lambda_{blockwork} = 0.4$ ($f_{diff\ max}$) A = 38 m^2
5. blockwork $\lambda_{blockwork} = 0.2$ ($f_{diff\ avg}$) A = 55 m^2
6. blockwork $\lambda_{blockwork} = 0.2$ ($f_{diff\ avg}$) A = 38 m^2
7. brickwork
8. blockwork $\lambda_{blockwork} = 0.0$ ($f_{diff\ min}$)

The one exception is that the curve for brickwork crosses the blockwork curves as W increases. For wider wall panels, brickwork becomes relatively stronger than most blockwork supporting a dead load.

In considering failure of the cavity wall by the two leaves failing independently, as discussed in Section 9.1.4, a blockwork leaf supporting minimal dead load is slightly weaker than the brickwork leaf. Otherwise, in most cases, the two leaves would fail at approximately the same ΔP. In extreme cases, f_{diff} up to 0.5 m higher may be required for the blockwork panel to fail. Because the failure pressures are so close, situations would be rare in which only one leaf fails. These pathways in Figures 9.1.4.1 and 9.1.4.2 are unlikely. An acceptable interpretation would be a binary system in which the wall panel either does not fail or fails entirely.

9.5.2 Strengthening Wall Panels

As noted in Section 9.4.3, the most effective means of strengthening a wall panel is increasing ρ or þ of the materials used. Bricks may be difficult to change because their size and material are ingrained in current practice (e.g. BS 3921 : 1985). Conversely, blocks are available in many sizes and densities (see BS 6073-2 : 1981 and Tables 9.3.1.2 and 9.3.1.3). Alternatively, two courses of each material could be used for constructing a cavity wall. Increasing þ without altering ρ has the advantage of increased stability along with the advantage of increased self-weight, perhaps reflected in þ's significant ability to change a wall panel's strength.

Caution must exercised in recommending a specific solution tailored to flood management without considering the wider context. Increasing either ρ or þ increases the amount of material used with detrimental environmental and economic consequences. Furthermore, changing only one leaf would not necessarily help in preventing the other leaf from collapsing, given the uncertainties in the failure mechanism.

Another option is to increase the effective þ of the wall panel by using ties which completely and unambiguously transfer load between panels. The issue of effective þ is tricky (leading to the conservative assumption in these calculations that the ties do not transfer load):

- Foster (2000) writes "The lateral stiffness of the whole [cavity] wall is less than that of a solid wall equal in thickness to the sum of the thicknesses of the two leaves and this must be taken into account in the determination of wall thicknesses".
- Hendry *et al.* (1981) states "The effective thickness for cavity walls, is taken as the greater value of two thirds the sum of the actual thicknesses of the two leaves or the actual thickness of the thicker leaf".
- West *et al.* (1977) suggest "Generally with vertical twist strip ties the moment of resistance of the cavity wall may be taken as the sum of the moments of resistance of the two leaves".

These authors do not justify their statements. A feasible option is to mandate the use of ties which would definitely produce an effective þ equal to the sum of the thicknesses of the two leaves. Care would be required to ensure that the ties are designed without exacerbating other problems, such as condensation wetting the masonry.

In any case, the open cavity wall is already uncommon, with the norm being cavities partially or completely filled with insulation. Whether or not this insulation could transfer loads or increase a leaf's effective thickness depends on the material type and amount used. Figures 9.4.3.3 and 9.4.3.4 illustrate the effects on panel strength of changing þ, suggesting that the value used for a panel's effective þ may significantly affect the f_{diff}-v combinations which would cause failure.

Decreasing wall panels' effective height would strengthen them. A strong, rigid connection between the panels half-way up each storey could act as a continuous or simply supported edge. Considering a continuous edge, reducing H from 2.4 m to 2.1 m had inconsequential effect on the wall panel's strength (Tables 9.4.3.1 and 9.4.3.2), but using H = 1.2 m may increase failure f_{diff} by up to 2.0 m depending on the other parameters. Achieving this connection without sacrificing the function of the cavity or the position and size of openings would be a design challenge.

The most effective means of strengthening a wall panel would be to increase ρ or þ of the materials used and the most effective means of weakening a wall panel would be to decrease these parameters' values. Despite the environmental and economic costs of overusing

material, wall panels are already relatively weak under lateral pressures from floods. Any movement towards reducing ρ or þ for wall panels in residences should be avoided.

9.6 Critique

This analysis assumed that the yield line pattern is shown in Figure 9.2.1.1. Although similar patterns are observed empirically in blockwork and brickwork (e.g. CERAM, c. 2001; Easterbrook, 2001; Templeton *et al.*, 1988a), other patterns are observed and postulated. An inverted-Y (Figure 9.6.1) and Martini's (1998) pattern (Figure 9.6.2) are possibilities. Sinha (1978) also considers the pattern in Figure 9.6.3, but the hydrostatic pressure component in floods may preclude such a pattern.

Alternative Patterns for Yield Line Analysis

Figure 9.6.1

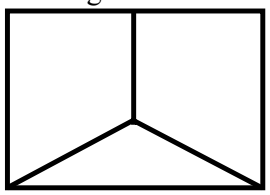


Figure 9.6.2

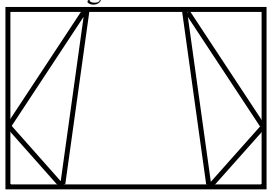
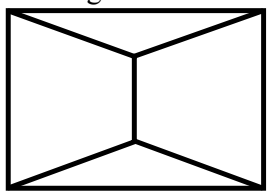


Figure 9.6.3



The hydrostatic pressure component influences the pattern seen. For example, in Figure 9.2.1.1, φ is introduced rather than the assumption of vertical symmetry which is used for uniform load analysis. Other patterns should be assumed for different support conditions. Hendry (1981) provides different patterns for different support conditions under uniform loads. Although the yield line pattern selected for this study is an assumption, a choice must be made and the choice made is reasonable for the conditions encountered and the problem defined. For the first-order approximation required for these calculations, the choice is adequate.

The second major limitation of these results is that the wall panels considered do not have openings. Many wall panels lack openings, hence these results are applicable to some real cases. Openings may weaken a panel by creating a hole through a solid mass and by providing points for failure to begin. Alternatively, door and window frames bridge the cavity, joining the two leaves in a manner suggested in Section 9.5.2. Front doors are generally 2.0 m high in a 2.4 m high wall panel. The door's effect could be to split the panel, reducing W and increasing the panel's strength. Similarly, large windows may produce several smaller, stronger panels rather than weakening the single, large panel.

Finally, the failure mechanism postulated in Section 9.1.4 may be incorrect. If the ties generally do transfer load, then walls would be stronger than suggested in this Chapter. Figures 9.4.3.3 and 9.4.3.4 permit estimations of panel strength for different effective panel thicknesses. Whether or not a yield line forms at the first floor in the outer leaf influences the results negligibly, but may nudge wall strength up or down depending on W (Table 9.6.4). The results arise because H/W, in addition to H and W, influences failure ΔP.

Table 9.6.4: Effect of Yield Line Forming at First Floor When v = 0 m/s
(Using A = 55 m^2)

j	Outer Leaf Description	H (m)	Failure f_{diff} (m)
	W = 2.0 m		
2	Yield line forms at the first floor, so H spans one storey.	2.4	1.1
2	Yield line does not form at the first floor, so H spans the residence.	4.8	1.1
3	Yield line forms at the first floor, so H spans one storey.	2.4	1.3
3	Yield line does not form at the first floor, so H spans the residence.	7.2	1.4
4	Yield line forms at the first floor, so H spans one storey.	2.4	1.5
4	Yield line does not form at the first floor, so H spans the residence.	9.6	1.6
	W = 5.0 m		
2	Yield line forms at the first floor, so H spans one storey.	2.4	0.9
2	Yield line does not form at the first floor, so H spans the residence.	4.8	0.9
3	Yield line forms at the first floor, so H spans one storey.	2.4	1.1
3	Yield line does not form at the first floor, so H spans the residence.	7.2	1.1
4	Yield line forms at the first floor, so H spans one storey.	2.4	1.3
4	Yield line does not form at the first floor, so H spans the residence.	9.6	1.2

9.7 Conclusions

Much research into unreinforced masonry collapse under lateral pressures has yet to be completed and caution is suggested in interpreting some of the results presented. Despite the assumptions made and the uncertainties, the method used is validated by comparison with the literature and it produces results based on a plausible failure mechanism. The goal of obtaining first-order results was achieved and uncertainties were analysed. The outcome is that a preliminary understanding of the strength of residence walls, and of strengthening them, has been achieved.

Many residence walls would collapse when $f_{diff} \geq b + 1.0$ m with $v = 0.0$ m/s. With $v > 0.0$ m/s, f_{diff} for collapse can drop below $b + 0.5$ m for some wall panels. These results add evidence to support the recommendations from DTLR (2002) and EA/CIRIA (2001) that homeowners should not seal their properties to a level above 0.9-1.0 m above ground level. The most effective means of strengthening walls is to increase ρ or þ for brickwork and blockwork. Nonetheless, reasons unrelated to flood actions must be considered for formulating policy on strengthening residence walls.

The results presented indicate the lowest ΔP at which the panel would definitely fail, not the highest ΔP at which the panel would definitely survive. This upper bound provides the information and tools for wall failure analysis of specific residences and residence classes which can be applied to creating vulnerability matrices (Chapter 12).

10. Glass Failure

The vocabulary used in this Chapter and the rest of this dissertation (McEvoy, 2001-2) is that:

- "pane" refers to a single sheet of glass.
- "unit" refers to all the glass used in a window or door.

A single-glazed unit has one pane while a double-glazed unit has two panes. Mentioning "pane" or "unit" implies that glass is the material. At times, any of "glass", "pane", or "unit" would be appropriate.

10.1 Theoretical Justification

10.1.1 Introduction

Soda-lime glass is generally used in residential units in England (ISE, 1999; Field, 2001). According to commercial glazing companies in England, for example The Window Man and The BB Group, along with commercial literature, for example from the U.K. Glass and Glazing Federation, glass in residences is almost always "normal strength"; i.e., not treated to change its strength. Some national lobby groups are trying to change this situation, such as the Laminated Glass Information Centre. The contemporary situation, though, means that examining normal-strength, soda-lime glass would be appropriate for this study.

10.1.2 Past Studies

BS 6262 : 1982 is the British Standard which provides design guidance for glass in buildings subject to uniform wind loading. The wind load is the 3 s gust for pressure and suction. All four edges of the pane are assumed to be supported. Knowing the type of glass, the wind load, the glass pane's area (R), and the ratio of the two sides (H/W or W/H), the glass thickness ($þ_{glass}$) is dictated. The parameter sets considered are:

- type of glass:
  - single-glazed: normal glass, wired glass (polished and cast), rough cast and patterned glass, laminated glass, toughened glass
  - double-glazed: normal glass, units with different $þ_{glass}$ for each pane, toughened glass, laminated glass
- wind load: 0.6 kPa to 5 kPa continuously
- R: 0.4 m^2 to 20 m^2 continuously

- H/W or W/H (the result must be greater than 1): 1 to 3 continuously
- $þ_{glass}$: 3 mm to 15 mm with discrete intervals of varying size

The log-log charts in BS 6262 : 1982 graph R versus uniform ΔP in a straight line representing the H/W or W/H ratio which is acceptable for a given $þ_{glass}$. Sample results are extracted in Table 10.1.2.1 for single- and double-glazed normal glass.

Table 10.1.2.1: Sample Values Extracted from Figures 7 and 10 in BS 6262 : 1982 (The H/W or W/H and $þ_{glass}$ values reported are those at the point of intersection of the given R and ΔP. When this point falls outside the bands which yield definite results, the H/W or W/H ratio is not given and the nearest higher $þ_{glass}$ is suggested.)

R (m^2)	Uniform ΔP (kPa)	single-glazed		double-glazed	
		H/W or W/H	$þ_{glass}$ (mm)	H/W or W/H	$þ_{glass}$ (mm)
0.5	1	--	3	--	3+3
1	1	--	3	--	3+3
2	1	--	4	--	3+3
5	1	1.5	6	2.5	4+4
10	1	2.0	10	1.5	6+6
0.5	2	--	3	--	3+3
1	2	--	4	--	3+3
2	2	2.5	5	--	4+4
5	2	2.0	10	1.5	6+6
10	2	--	15	2.0	10+10
0.5	4	--	4	--	3+3
1	4	2.5	5	--	4+4
2	4	--	10	2.5	5+5
5	4	1.0	15	2.0	10+10
10	4	3.0	19	--	> 10+10

British Standards detailing strength tests for glass, panes, or units include BS 6206 : 1981, BS 6262-4 : 1994, BS 6375-2 : 1987, and BS 7950 : 1997. These tests do not apply to the flood actions considered here because the loading mechanism is different. They have nevertheless been useful for indicating the approximate expected strength of units (see Chapter 7).

Beason and Morgan (1984) present experimental data from testing units to failure (Table 10.1.2.2). Simply supported edges and constant, uniform, lateral loading are implied, although these details are not stated explicitly.

Table 10.1.2.2: Empirical Test Results from Beason and Morgan (1984)

sample #	dimensions	failure data	k for Wei(ξ) (Section 10.1.4)
1	0.724 m × 1.540 m × 5.56 mm	3.78 kPa ± 0.88 kPa	6
	sample size 20, glass age 20+ years		
2	0.724 m × 0.724 m × 5.56 mm	8.05 kPa ± 1.80 kPa	6
	sample size 20, glass age 20+ years		
3	0.413 m × 0.502 m × 3.18 mm	10.99 kPa ± 2.97 kPa	6
	sample size 22, glass age ≈20 years		
4	0.356 m × 0.921 m × 3.18 mm	6.43 kPa ± 1.61 kPa	5
	sample size 132, glass age ≈25 years		
5	0.413 m × 0.502 m × 3.18 mm	20.49 kPa ± 3.71 kPa	9
	sample size not given, new glass		

For another comparison, Beason and Morgan (1984) projected the parameters calculated from the experimental data onto standard unit sizes in order to calculate ΔP for failure probabilities of 0.008 and 0.500 (Table 10.1.2.3). $þ_{glass}$ = 5.56 mm was assumed.

Table 10.1.2.3: Calculated Results from Beason and Morgan (1984)

unit size →	1.22 m × 1.22 m		1.52 m × 1.52 m		1.83 m × 1.83 m	
failure probability →	0.008	0.500	0.008	0.500	0.008	0.500
samples 1 and 2	1.22 kPa	3.65 kPa	0.85 kPa	2.57 kPa	0.65 kPa	1.91 kPa
sample 3	1.13 kPa	4.12 kPa	0.76 kPa	2.84 kPa	0.57 kPa	2.08 kPa
sample 4	1.45 kPa	4.38 kPa	1.03 kPa	3.07 kPa	0.79 kPa	2.27 kPa
sample 5	4.62 kPa	9.22 kPa	3.30 kPa	6.33 kPa	2.45 kPa	4.61 kPa
industry literature	2.54 kPa	no value	1.77 kPa	no value	1.29 kPa	no value
industry literature	3.21 kPa	8.02 kPa	2.06 kPa	5.15 kPa	1.44 kPa	3.59 kPa

So and Chan (1996) report test results obtained from the National Research Council of Canada from 47 simply-supported, in-service units subjected to a constant, uniform, lateral load. Dimensions were 1.3-1.4 m × 0.9-1.3 m with $þ_{glass}$ ≈ 4 mm. Failure ΔP range from 2 kPa to 6 kPa. Sample results are in Table 10.1.2.4. So and Chan (1996) compare their results to a nonlinear finite element model based on thin plate analysis; i.e. þ << H,W for each pane. The model calculates failure ΔP for a failure probability of 0.008 and produces failure ΔP approximately 2.5 times lower than the empirical results.

Table 10.1.2.4: Sample Empirical Test Results from So and Chan (1996)

Sample	Dimensions	failure ΔP (kPa)
1	1.300 m × 0.905 m × 4.10 mm	2.84
15	1.300 m × 0.925 m × 3.95 mm	4.79
22	1.300 m × 0.900 m × 4.00 mm	2.92
26	1.300 m × 0.975 mm × 3.96 mm	6.18
41	1.374 m × 1.342 m × 3.83 mm	1.86
43	1.300 m × 1.062 m × 3.94 mm	3.27

Vallabhan (1983) compares a finite difference method with other numerical techniques for analysing simply-supported, rectangular units under uniform, lateral loads. All methods suggest that large deflection theory is needed. One of the numerical examples given involves a unit with dimensions 2.438 m x 1.524 m x 5.7 mm which is subjected to a slowly-increasing ΔP. The deflection theoretically reaches almost 50 mm at a pressure of 5.5 kPa, although it appears that no option exists in the software for the glass to fail.

West (1973) reports experimental results for timber-framed units subjected to explosion pressures. Such pressures have short-lived peaks, thereby differing from the hydrostatic and hydrodynamic lateral loading from floods which are more constant with time. West's (1973) results nevertheless indicate the disparity which can arise (Table 10.1.2.5). West (1973) also observes "distortion of the frame can cause the glass to break at pressures less than its actual breaking strength".

Table 10.1.2.5: Sample Empirical Test Results from West (1973)

Glazing	Dimensions ($þ_{glass}$ Not Given)	Test Results
single	1.02 m × 1.02 m	3.10 kPa, unbroken
single	1.02 m × 1.02 m	2.96 kPa broken
single	0.320 m × 0.973 m	11.03 kPa, broken
single	0.397 m × 0.576 m	11.03 kPa, unbroken
double	1.02 m × 1.02 m	6.00 kPa, broken
double	1.02 m × 1.02 m	6.00 kPa, unbroken

10.1.3 Thin Plate Theory

One relatively common theme compiled from the past studies reported in Section 10.1.2—as well as from Calladine (2001), ISE (1999), Vallabhan *et al*. (1993), and Vallabhan *et al*. (1994)—is that thin plate theory under large deflections with probabilistic failure is appropriate for window and door glass failure under a lateral load. From these references, the

following assumptions are chosen for examining unit failure in residence doors and windows under lateral flood loads:

1. The glass is represented as thin, flat plates (panes), i.e. $þ_{glass}$ << H,W. Panes cannot be represented as either beams or infinite plates because W/H ≈ 1.
2. The glass is a linearly elastic, brittle material.
3. The glass is isotropic, i.e. $E_x = E_y$.
4. The panes are rectangles simply supported on all four edges, i.e. M = 0 at the edges.
5. The panes undergo large deflections before breaking, i.e. δ is not << $þ_{glass}$ at failure, and in-plane deformations and stresses occur. Vallabhan *et al*. (1994) note that "Thin-glass window plates in buildings may undergo deflections 10 times their thickness before they fail by brittle fracture".
6. Failure is due to either M_x or M_y leading to bending stresses which overcome the glass' tensile strength. Failure is not due to shearing stresses.

10.1.4 Weibull Failure Probability

For failure, glass' material property of flexural or tensile strength (Ω) must be exceeded by the calculated maximum bending stress (σ_{max}) in the glass due to an applied ΔP. Uncertainty in this comparison occurs because microscopic flaws in glass lead to failure at stress values much lower than the theoretical Ω. The mechanism of failure is that microscopic flaws act as stress concentrators under load. The concentrated stress causes the cracks to enlarge and to propagate through the glass which leads to failure when large enough cracks have formed. More discussion of these flaws and their connection with Ω is reserved until Section 10.3.1 under the selection of Ω.

The Weibull distribution $Wei(\xi) = k\xi^{k-1}\exp(-\xi^k)$ for determining glass failure probability incorporates the randomness of such flaws to a degree and is appropriate for predicting unit failure (Beason and Morgan, 1984; Creyke *et al*., 1982; ISE, 1999; So and Chan, 1996). ξ > 0 is the variable and the shape parameter k > 1 must be defined. The integral of Wei(ξ) with respect to ξ is $1 - \exp(-\xi^k)$ which is interpreted as the cumulative failure probability (Figure 10.1.4.1).

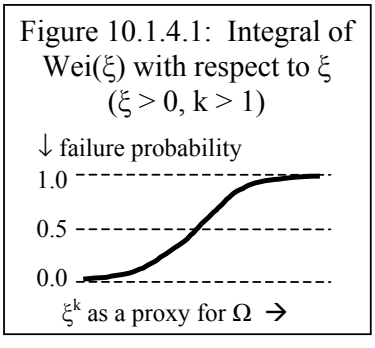


Figure 10.1.4.1: Integral of Wei(ξ) with respect to ξ (ξ > 0, k > 1)

Beason and Morgan (1984) interpret ξ^k as a risk function defined as a product of hazard, including ΔP, and vulnerability, incorporating the pane's geometry. Based on empirical data, they further suggest values of k between 5 and 9 (Table 10.1.2.2).

Creyke *et al.* (1982) propose ξ as a function of three parameters:

- Ω.
- σ for which the failure probability is 0: the researcher either needs "to guess at a value" or to set it to 0.
- "a normalising factor which bears no relationship to an obvious measurable property".

k and the normalising factor are determined by testing several samples and plotting experimental Ω against failure probability. Without justification, the failure probability for each experimental Ω is assigned "a nominal probability of failure" equal to the ordinal rank of that Ω value divided by the sample size plus one.

In place of ξ^k in the Weibull equation, So and Chan (1996) suggest:

$$(R_{pane} / 1.0\ m^2) \times [S / (32.1\ MPa)]^k$$

Based on empirical data, they propose:

- k = 7 is still termed a Weibull parameter.
- S is the individual pane's characteristic strength and is a function of k = 7, ΔP, and W/H.
- 1.0 m^2 represents R for a reference pane.
- 32.1 MPa represents the characteristic Ω for a reference pane.

These three examples display the variety of approaches taken by authors. The decision of how to develop a specific Wei(ξ) is somewhat arbitrary according to the needs of a particular application. Each author selects their own approach in the context of their needs.

10.1.5 Conclusions

For this analysis of window and door units failing under lateral flood loads, using the physical properties of normal-strength soda-lime glass is appropriate. The theory involves thin plates undergoing large deflections and probabilistic failure according to the Weibull distribution. The specific interpretation of the Weibull distribution depends on the specific needs of each problem. Empirical evidence is available for only uniform loads on mainly single-glazed units. Failure ΔP is generally between 1 kPa and 10 kPa although most tests use R > 1.00 m^2.

Smaller panes generally have failure $\Delta P > 10$ kPa. Translating the uniform load results into a hydrostatic plus hydrodynamic pressure is not straightforward, but a rough estimate would be that failure would occur when the linear load's ΔP_{avg} approximately equals the uniform failure ΔP.

10.2 Calculations

10.2.1 Introduction to Aalami and Williams (1975)

Aalami and Williams (1975) provide comprehensive data on thin plates under various lateral loads by providing computer-calculated graphs of dimensionless σ_{max} versus dimensionless ΔP. "Pane" is used instead of "plate" for this discussion. The assumptions in Section 10.1.3 apply to their work. Under defined pane and load conditions, they provide σ_{max} over the entire pane, both the spatial maximum and the maximum of the stresses in the x and y directions.

10.2.2 Uniform Load

For uniform loads, the graphs used from Aalami and Williams (1975) are "rigidly supported, rotationally free" for the flexural boundary conditions and "zero direct stress, zero shear stress" for the membrane boundary conditions. These boundary conditions apply to the pane's four edges. Aalami and Williams (1975) consider $H_{pane}/W_{pane} = 1.0, 1.5, 2.0, 3.0$. Symmetry yields $H_{pane}/W_{pane} = 0.33, 0.50, 0.67$.

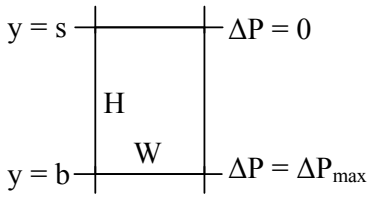
The maximum bending stresses for uniform loads appear near, but not at, the pane's centre (Aalami and Williams, 1975). Vallabhan *et al*. (1994) comment "As the nonlinearity in the plate increases, the maximum tensile stress in the plate, which is maximum at the [centre] of the plate for low-lateral pressures, moves closer to the edge of the plate".

10.2.3 Linear Load

Aalami and Williams (1975) provide results for only linear loads which assume that $\Delta P = \Delta P_{max}$ at $y = b$ and $\Delta P = 0$ at $y = s$ (Figure 10.2.3.1). Graphs are not provided for the four pressure distributions in Figure 3.2.2.1. Graphs are provided for flexural boundary conditions which are "rigidly supported, rotationally free" on the pane's four edges, but only for

membrane boundary conditions which may not apply to panes in residence units. These membrane boundary conditions are "edge remains straight, zero average direct stress, zero shear stress" on three of the pane's edges and "zero extensional displacement, zero shear stress" on the three of the pane's edges. In each case, the forth edge, which is at y = s (i.e. where $\Delta P = 0$), has membrane boundary conditions which are "zero direct stress, zero shear stress". These two graphs must be used as approximate values. The difference between the two graphs is a maximum of 15%, and is usually much less, hence it appears as if different pane boundary conditions provide similar results for linear loads.

Figure 10.2.3.1: Aalami and Williams' (1975) Pressure Distribution for Linear Loads



The conditions considered are H_{pane}/W_{pane} = 0.67, 1.0, 1.5, 2.0. Symmetry does not apply for other ratios, but interpolation and extrapolation will be used.

10.2.4 Calculation Method

As per the assumptions made in Chapter 3, and given the graphs available from Aalami and Williams (1975), situation (a) in Figure 3.2.2.1 is considered, i.e. f_{diff} and v occur instantaneously at a previously dry residence and $f_{diff} \geq s$. σ_{max} for this ΔP function is calculated through superposition of a hydrostatic load and a uniform load (Roarke and Young, 1975). One concern is that σ_{max} occurs at different locations in the pane for each of the hydrostatic load and the uniform load. Thus, finding the overall σ_{max} for the superposition is not feasible with the available data. The solution is the conservative approximation that the two maxima occur close enough to each other that they may be superposed.

The basic pressure equation is:

$$\Delta P = \rho_w g(f_{diff} - y) + \tfrac{1}{2}\rho_w v^2 = \Delta P_{y=0} - \rho_w g y \text{ for } b \leq y \leq f_{diff}$$

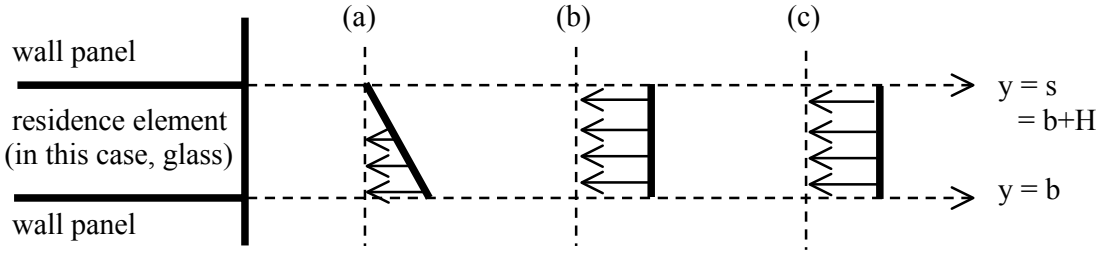
$$\Delta P = 0 \text{ for } y > f_{diff}$$

From this equation, three pressures are derived which must be superposed for calculating unit failure (Figure 10.2.4.1):

(a) ΔP due to f_{diff} acting directly on the unit: a linear, hydrostatic component with $\Delta P_{y=b} = \rho_w g H$ and $\Delta P_{y=s} = 0$.

(b) ΔP due to v acting directly on the unit: a uniform hydrodynamic component with $\Delta P_{b<y<s} = \tfrac{1}{2}\rho v^2$.

Figure 10.2.4.1: Superposition for Combined Hydrostatic and Hydrodynamic Load



(c) ΔP due to f_{diff} above the unit: a uniform hydrostatic component with $\Delta P_{b<y<s} = \rho_w g(f_{diff} - s)$.

Using the graphs in Aalami and Williams (1975) of dimensionless σ_{max} versus dimensionless ΔP the steps below were followed. Linear load refers to component (a) in Figure 10.2.4.1 while uniform load refers to components (b) and (c) in Figure 10.2.4.1.

1. A specific pane is defined by selecting H, W, and þ.
2. The choice of H defines $\Delta P_{y=b}$ for the linear load. From H, W, þ, and $\Delta P_{y=b}$, the graphs yield σ_{max} from the linear load.
3. σ_{max} from the uniform load depends on ΔP from the combination of f_{diff} and v. A matrix of f_{diff}-v pairs for $f_{diff} > s$ is created. ΔP is calculated for each f_{diff}-v pair. The graphs are used to determine σ_{max} from the uniform load for each f_{diff}-v pair. The result is a σ_{max} matrix with each σ_{max} entry representing an f_{diff}-v pair.
4. σ_{max} from the linear load (determined in step 2) is added to each σ_{max} in the matrix to yield an overall σ_{max} matrix experienced by the pane under each f_{diff}-v pair.
5. In Section 10.3.1, Ω and Wei(ξ) are used to determine the σ_{max} values at which certain failure probabilities would occur. These values are compared with σ_{max} experienced by the pane (from step 4) to determine the failure probability of the pane defined in step 1. A probability failure matrix results. Given an f_{diff}-v pair, the probability of glass failing is provided.

For double-glazed units, which are the dominant units in the case study sites (Chapter 6), this procedure requires one extra step related to the ΔP value used to calculate σ_{max}. For double-glazed units, FACETS (2001) states "As the cavity of insulated units is sealed, deflection of one pane of a unit under applied load will affect the pressure in the cavity and apply a load to the other pane. The load will therefore be shared by the panes in proportion to their stiffness". Chisholm (2001) and Keiller (2001) agree that although one pane would likely take more load

than the other, a reasonable approximation is that each pane would take half the load. ISE (1999) asks whether load sharing occurs and comments "In general the answer is yes, though it is not so for small stiff panes of glass separated by deep cavities". Therefore, for double-glazed units, $\Delta P/2$ will be used to calculate σ_{max} from the graphs in Aalami and Williams (1975). Approximately twice as much load must be imposed on a double-glazed unit for σ_{max} to reach the same value in each pane experienced by a single-glazed unit.

The steps outlined here are the theoretical ideal procedure. Modifications were required given the data available, as will be noted in Section 10.3 and the results provided in Section 10.4.

10.3 Parameters

10.3.1 Selecting Parameters and Glazing Properties

To find the failure probability for units, the parameters described in this Section are needed. Each parameter is presented with the value chosen followed by the sources which contributed to the selection of that value. The decisions and parameter names are summarised in Section 10.3.2. The sensitivity of the results to parameter choices is examined in Section 10.4.3.

$b = 0$

b does not yet need to be considered. The calculations are presented for f_{diff} relative to the unit, not relative to the ground. To interpret the results, b is needed, hence discussion of this parameter appears in Section 10.5.2.

E = 73 GPa

From Table 10.3.1.1, a wide range is feasible but 70-75 GPa encompasses most previous work.

Table 10.3.1.1: Young's Modulus for Glass

Source	Value and Comments
Bansal and Doremus (1986)	54 GPa, 73 GPa, 73 GPa, and 78 GPa for four types of optical glasses. For vitreous silica at 25°C, dE/dPressure = -3.55.
Doremus (1973)	72 GPa for silica glass.
Efunda (2001)	48-83 GPa is the range for glass.
ISE (1999)	70-74 GPa is a typical value for annealed glass.
Makovička (1998)	60 GPa for window glass broken by flexure.
Persson (1983)	70-80 GPa for window glass
Vallabhan (1983) and Vallabhan *et al*. (1993)	68.9 GPa, because this value corresponds to the round number of 10^7 pounds per square inch.
Vallabhan *et al*. (1994)	70 GPa for window glass.

$g = 9.80665\ m/s^2$ (Lide, 2001).

Pane shape and dimensions were determined in conjunction with carrying out the residence survey (Chapter 6). Fewer than 1% of window units and fewer than 5% of door units were non-rectangular. The non-rectangular door panes tended to be semi-circles or portions of semi-circles with small dimensions ($R < 0.05\ m^2$). Preliminary analyses indicated that units of such small R would have a negligible failure probability well above ΔP expected to cause catastrophic failure in wall panels (Chapter 9).

For H and W, it soon became apparent that unit sizes occurred in a wide array of dimensions with H,W < 2.0 m. For a usable analysis, some of form of grouping would be required. Two groups emerged. Significant numbers of door units had H = 2.0 m and 0.7 m < W < 1.1 m. From the remaining units, covering both doors and windows, more than 98% had H,W < 1.1 m.

Two criteria were important for selecting unit classes: a reasonable number of units for analysis balanced with classes disparate enough to be rapidly and accurately identified in the field. Choosing intervals for H and W of 0.25 m was felt to best balance these criteria, even though measurement accuracy to two decimal places is unrealistic. Thus, unit dimensions studied were:

- H,W $\in$ {0.25 m, 0.50 m, 0.75 m, 1.00 m} for sixteen possible panes.

- (H,W) = (2.00 m, 0.75 m)
- (H,W) = (2.00 m, 1.00 m)

The total was eighteen different units for analysis. Although H and W are reported to two decimal places, the values given represent the range around each value. This approach to selecting unit sizes has the advantages of rapid assessment during the residence survey along with efficient, but usable, analysis.

$\nu = 0.3$

From Table 10.3.1.2, $0.22 \leq \nu \leq 0.25$ would be the most appropriate choice. The graphs in Aalami and Williams (1975) assume $\nu = 0.3$, so the only option is to assume that value. Aalami and Williams (1975) point out that if $\nu = 0.3$ is assumed but in reality $0.2 < \nu < 0.27$, then errors in σ_{max} would be 1-3%. Therefore, $\nu = 0.3$ is an appropriate assumption.

Table 10.3.1.2: Poisson's Ratio for Glass

Source	Value and Comments
Bansal and Doremus (1986)	0.21, 0.22, 0.25, and 0.27 for four types of optical glasses.
Efunda (2001)	0.2-0.27 is the range for glass.
ISE (1999)	0.22 is a typical value for annealed glass.
Makovička (1998)	0.25 for window glass.
Persson (1983)	0.25 for soda-lime-silica glass.
Vallabhan (1983), Vallabhan *et al*. (1993), and Vallabhan *et al*. (1994).	0.22 for window glass

$\rho_w = 1{,}026$ kg/m^3 (Lide, 2001).

Ω must be discussed in the context of σ_{max} and failure probability because the understanding of Ω is inherently linked to them. Theoretical tensile strengths of glass are not useful for glass failure, because failure is fracture-based resulting from microscopic flaws on the glass surface which are unavoidable in a practical situation (Lawn, 1993 and the references below):

- Doremus (1973) notes that "The theoretical strengths…are much greater than strengths actually measured for ordinary glasses which range from about $[3\times10^7]$ to 10^8 N/m^2. This reduction is caused by flaws in the surface of the glass, which concentrate applied stresses…The design strength of ordinary glasses is 500 to 1000 times lower than their theoretical strength".

- Persson (1983) writes that "The tensile strength of virgin soda-lime-silica glass is approximately [10 GPa]. In practice, however, glass products have a strength which is only 1% of this value". The actual strength depends on the nature of the chemical or mechanical attack producing the flaws in the glass.
- ISE (1999) explains that "for a brittle material like glass…the failure-causing flaw may not be located at the point of maximum tensile stress". For example, failure could occur due to fractures or flaws inside a pane rather than due to the surface tensile stresses.
- Uhlmann and Kreidl (1980) state that strengths of the order 10^4 MPa have been measured in pristine glass fibres but "strengths of glasses under most conditions are 10-100 times lower than this value" due to the unavoidable defects.

Table 10.3.1.3 lists values for the practical, not theoretical, strength of glass.

Table 10.3.1.3: Tensile Strength of Glass

Source	Value and Comments
Bansal and Doremus (1986)	70 MPa is typical in good conditions, but for design, < 20 MPa should be used. Pane sizes are not given.
Chen and Matsumura (1997)	The range is from 60 MPa causing failure in 1 s down to 37 MPa causing failure in 10^5+ s. Panes were 60 mm × 10 mm × 5 mm with scratches.
Creyke *et al*. (1982)	65 MPa is the typical bend strength of soda-lime glass. For abraded soda-lime glass, the range of bend stress versus time to failure is from 60 MPa in 10^{-2} s to 20 MPa in 10^5 s.
Doremus (1973)	30-100 MPa for ordinary glass.
Hamidouche *et al.* (1994)	87.4-98.3 MPa depending on the method used for fracturing. Samples were 40 mm × 6 mm × 4 mm with deliberate scratches.
Lawn (1993)	The maximum possible tensile strength of glass specimens is 170 MPa.
Li *et al.* (1995)	50-80 MPa for a stressing rate up to 25 MPa/s for soda-lime glass rods of 3 mm diameter and 60 mm length.
Makovička (1998)	The mean medium-term breaking strength of glass is 55 MPa. The mean probable strength is 33 MPa. The mean long-term strength is 13 MPa. The design strength to static load is 6.6 MPa.
Mazurin *et al.* (1983)	The bending strength of SiO_2 glass is 500-700 MPa for fibre diameters of 0.05-0.1 mm. The mean impact strength of SiO_2 glass is 83 kPa at room temperature.
McCarthy and Rossell (1993)	55.6-115 MPa for deliberately scratched microscope slides of dimensions 77 mm × 26 mm × 1 mm with various coatings.
Persson (1983)	30-100 MPa for an impact on 4 mm sheet glass. The "maximum working stresses allowed for a risk of not more than 1% for a glass thickness up to 4 mm" for sheet glass is 15 MPa for sustained loading and 30 MPa for momentary loading.
So and Chan (1996)	32.1 MPa.

Uhlmann and Kreidl (1980)	30-80+ MPa for applied load times of 10^2 to 10^6+ s. For unstrengthened glasses in a load-bearing context, 14 MPa is the failure threshold.
West (1973)	"The Glazing Manual notes that there is considerable variation of strength in supposedly similar samples…the allowable maximum working stresses for sheet glass are [15.2 MPa] for sustained loads and [40 MPa] for momentary [3 s] loads. The latter is intended to represent the resistance to gusts of wind."

The values in Table 10.3.1.3 indicate that Ω's range in a practical setting would be between approximately 20 MPa and approximately 80 MPa. A typical or generally expected value would be 55-70 MPa.

Due to the uncertainties involved in the calculations and inconsistencies amongst references, a single, non-contentious Wei(ξ) could not be proposed. Section 10.1.4 provides references which have different interpretations of Wei(ξ). The commonality amongst the sources is that a calculated σ_{max} may be mapped onto, or interpreted as, a probability failure. Furthermore, as noted in Chapter 3, this study examines first-order vulnerability. A prudent approach therefore would be to suggest approximate mappings from σ_{max} to failure probability which are not only reasonable in the context of all previous studies but which also could be applied to the practical problem of glass failure analysed here. This approach emulates that of other studies, as noted in Section 10.1.4, in which a method is selected appropriate to the needs of the problem being solved.

When σ_{max} is calculated to be near the low end of the potential range for Ω, a low failure probability would be expected. When σ_{max} is calculated to be at the high end of the potential range for Ω, a high failure probability would be expected. For these reasons, the limits of Ω's identified range, 20 MPa and 80 MPa, could be interpreted as the limits of the σ_{max} range to be mapped onto reasonable failure probabilities.

The suggestion is that $\sigma_{max} \approx 20$ MPa could be interpreted as approximately a 1% failure probability and $\sigma_{max} \approx 80$ MPa could be interpreted as approximately a 99% failure probability. Given the asymmetry of Wei(ξ) and the values from Table 10.3.1.3, a further suggestion is that $\sigma_{max} \approx 60$ MPa could be interpreted as approximately a 50% failure probability. Mapping σ_{max} onto failure probabilities is used as a proxy for defining Ω. Chisholm (2001) and Keiller (2001) support this approach and the suggested selections in the context of this study and its specific goals.

An established, accepted method does not exist for checking the validity of these approximations, even with a specific Weibull distribution. As shown in Section 10.1.4, most authors make their own, often arbitrary choices which are certainly appropriate for their purposes. The approximations suggested here are similarly appropriate for the purposes here, but the results must be compared with the literature (Section 10.1.2) to determine the validity of these approximations. This comparison occurs in Section 10.5.

$þ_{glass}$ must be discussed in the context of the glazing used in residences in England. Almost all glazing in windows and doors in residences in the case study sites is double-glazed units with normal-strength glass and $þ_{glass}$ = 4 mm for each pane. These properties were decided by using information from Baird (2002), Chisholm (2001), Keiller (2001), and several commercial glazing companies in England including The Window Man, The BB Group, Hill Leigh Ltd., and Yehi. The following discussion relies on information from these sources but the analysis and decisions may not necessarily be attributable directly to these sources.

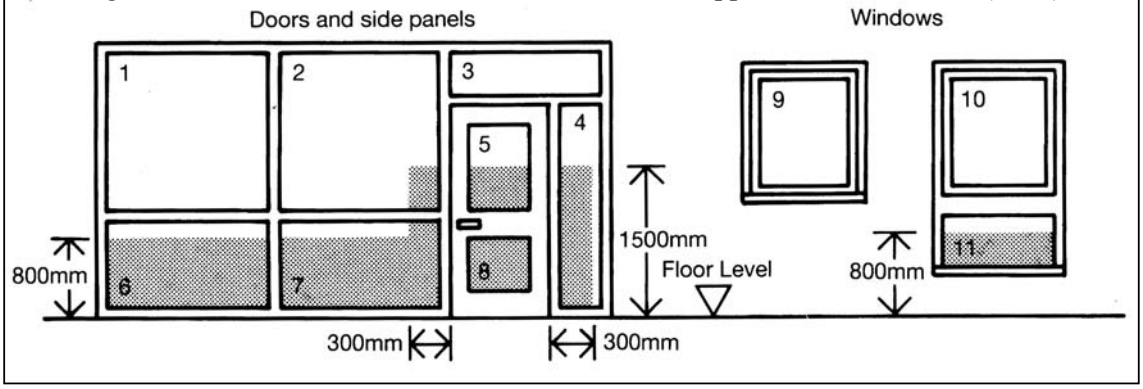
Chapter 6 confirms that most glazing in residences in the case study sites is double glazed. Therefore, only double-glazed units with $þ_{glass}$ = 4 mm for each pane, for both windows and doors, are considered in the glass failure calculations. Such an approach is acceptably robust. While single glazing is undergoing rapid replacement with standard double glazing, other products do not receive much attention. Triple glazing, laminated glass, thicker panes, and specially strengthened glass all exist and are promoted by firms. The uptake of these products in the standard residential market in England is low and is not expected to increase significantly in the near future.

Approximately 7% of units were frosted or had additional designs engraved. Due to this low percentage, such designs were not considered in the strength calculations. All panes were assumed to be smooth, flat plates of glass.

Critical locations, where special criteria for residence units apply (Figure 10.3.1.1), are another issue to consider. Units in critical locations must adhere to Requirement N1 in Approved Document N (1998):

> "(a) if broken on impact, break in a way which is unlikely to cause injury; or
> (b) resist impact without breaking; or
> (c) be shielded or protected from impact"

Figure 10.3.1.1: Critical Locations for Glass from Approved Document N (1998)



Small panes, defined as min(H,W) ≤ 0.25 m and R ≤ 0.5 m^2 are permitted with $þ_{glass}$ ≥ 6 mm. The option clearly exists to use stronger glass but standard industry practice for residences is to opt for (a), the safe breakage criterion.

BS 6206 : 1981 defines safe breakage. The classification of glasses and plastics according to "no breakage, or breaks safely" is based on a test involving a single impact of a concentrated load defined by energy terms. Such an approach does not apply to the flood actions considered here because those pressures are uniform and linear loads applied over a relatively long duration on the entire pane. In the absence of further safe breakage criteria, an appropriate and conservative assumption is that units in critical locations are not stronger than units in other locations. This assumption was confirmed as being appropriate by several commercial glazing companies in England.

10.3.2 Summary of Parameters

b (height of ground floor above ground level) = 0 to be generic, but results presented for f_{diff} must have b added just before interpretation in order to present the true situation. Further discussion appears in Section 10.5.2.

E (Young's Modulus) = 73 GPa

g (gravitational acceleration) = 9.80665 m/s^2

H and W are indicated in Table 10.3.2.1 and all panes are rectangular.

Table 10.3.2.1: H and W for the Analysis

Unit	H (m)	W (m)	R (m^2)	W/H
A	1.00	0.25	0.25	0.25
B	0.75	0.25	0.19	0.33
C	1.00	0.50	0.50	0.50
D	0.50	0.25	0.12	0.50
E	0.75	0.50	0.38	0.67
F	1.00	0.75	0.75	0.75
G	1.00	1.00	1.00	1.00
H	0.75	0.75	0.56	1.00
I	0.50	0.50	0.25	1.00
J	0.25	0.25	0.06	1.00
K	0.75	1.00	0.75	1.33
L	0.50	0.75	0.38	1.50
M	0.50	1.00	0.50	2.00
N	0.25	0.50	0.12	2.00
O	0.25	0.75	0.19	3.00
P	0.25	1.00	0.25	4.00
Q	2.00	1.00	2.00	0.50
R	2.00	0.75	1.50	0.38

ν (Poisson's ratio) = 0.3

ρ_w (water density) = 1,026 kg/m^3

Ω requires the proxy:

$\sigma_{max} \approx 20$ MPa implies a 1% failure probability

$\sigma_{max} \approx 60$ MPa implies a 50% failure probability

$\sigma_{max} \approx 80$ MPa implies a 99% failure probability

$þ_{glass}$ (glass thickness) = 4 mm for each pane

double-glazed units with smooth, flat panes and normal-strength glass

10.4 Results

10.4.1 Uniform Loads

Considering only uniform loads (Table 10.4.1.1), the principal trend is that failure ΔP increases as R decreases. Some of the weakest units, generally the largest ones, could break at $\Delta P < 5$ kPa. Once $\Delta P = 10$ kPa, all but the strongest units, generally the smallest ones, will

be failing. For $\Delta P = 20$ kPa, significant numbers of even the strongest units will be failing. For $\Delta P > 50$ kPa, few units would be left intact.

Table 10.4.1.1: Failure of Double-glazed Units Under Uniform Pressure

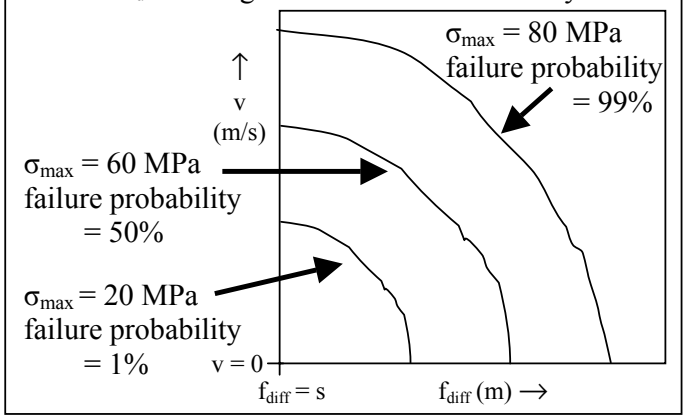
($þ_{glass} = 4$ mm. An approximate ranking of weakest to strongest units is given; see Section 10.5.1 for discussion)

Unit	H (m) and W (m) (no differentiation due to symmetry)	Uniform ΔP (kPa) for Failure Probability = 1%	= 50%	= 99%
Q	2.00×1.00	2	7	9
R	2.00×0.75	2	8	12
G	1.00×1.00	3	11	16
F, K	0.75×1.00	4	14	20
H	0.75×0.75	5	18	25
C, M	0.50×1.00	6	21	30
E, L	0.50×0.75	8	26	36
A, P	0.25×1.00	10	31	41
I	0.50×0.50	12	36	50
B, O	0.25×0.75	15	46	61
D, N	0.25×0.50	26	77	102
J	0.25×0.25	46	138	184

10.4.2 Flood Loads

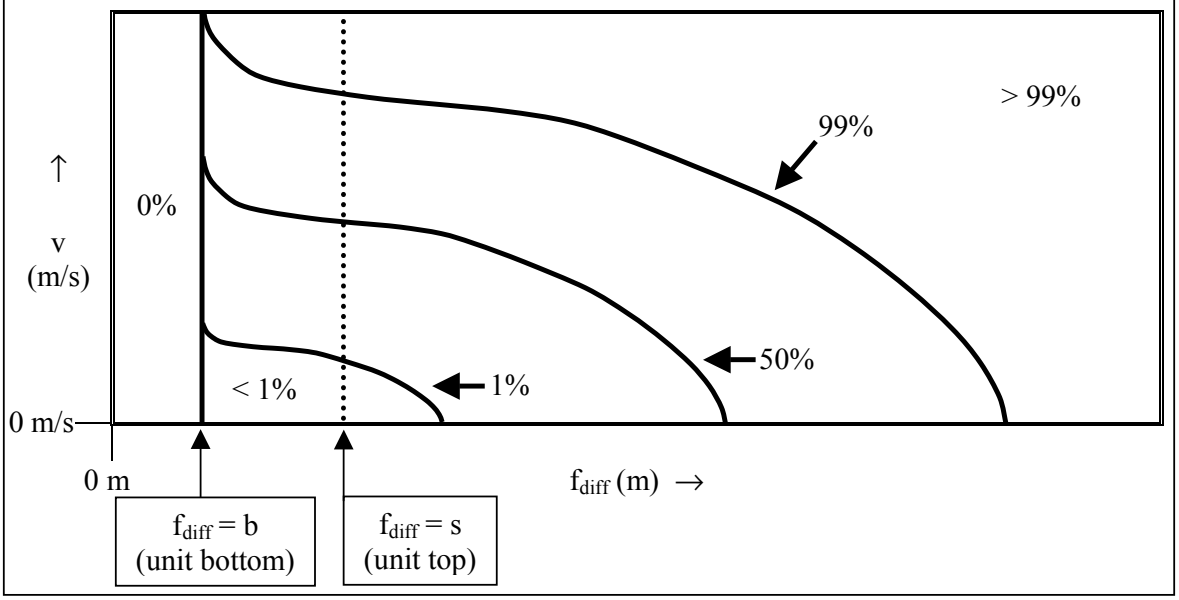
Using the procedure outlined in Section 10.2.4 with the parameters described in Section 10.3.2, f_{diff}-v graphs were produced. These graphs have three iso-σ_{max} curves for 20 MPa, 60 MPa, and 80 MPa corresponding to failure probabilities of 1%, 50%, and 99% respectively (Figure 10.4.2.1).

Figure 10.4.2.1:
f_{diff}-v Diagrams of Failure Probability



Only the f_{diff}-v region for $f_{diff} \geq s$, as illustrated in Figure 10.4.2.1, could be calculated. A theoretical, complete f_{diff}-v graph is provided in Figure 10.4.2.2.

Figure 10.4.2.2: Theoretical, Complete f_{diff}-v Graph
(Percentages represent glass failure probability.)



The five most useful f_{diff}-v graphs for determining vulnerability matrices (Chapter 12 as defined in Chapter 3) are provided as Figures 10.4.2.4 to 10.4.2.8. The reason why these figures cover only large units is discussed in Section 10.5.2. Only two iso-σ_{max} curves appear in each graph because σ_{max} = 20 MPa is reached for b < f_{diff} < s, off the scale of these graphs (Figure 10.4.2.3). Thus, these units have a non-negligible failure probability before f_{diff} = s; i.e., before the flood water has reached the unit's top.

Figure 10.4.2.3: Theoretical, Complete f_{diff}-v Graph for Large Units
(Schematic for Figures 10.4.2.4 to 10.4.2.8. Percentages represent glass failure probability.)

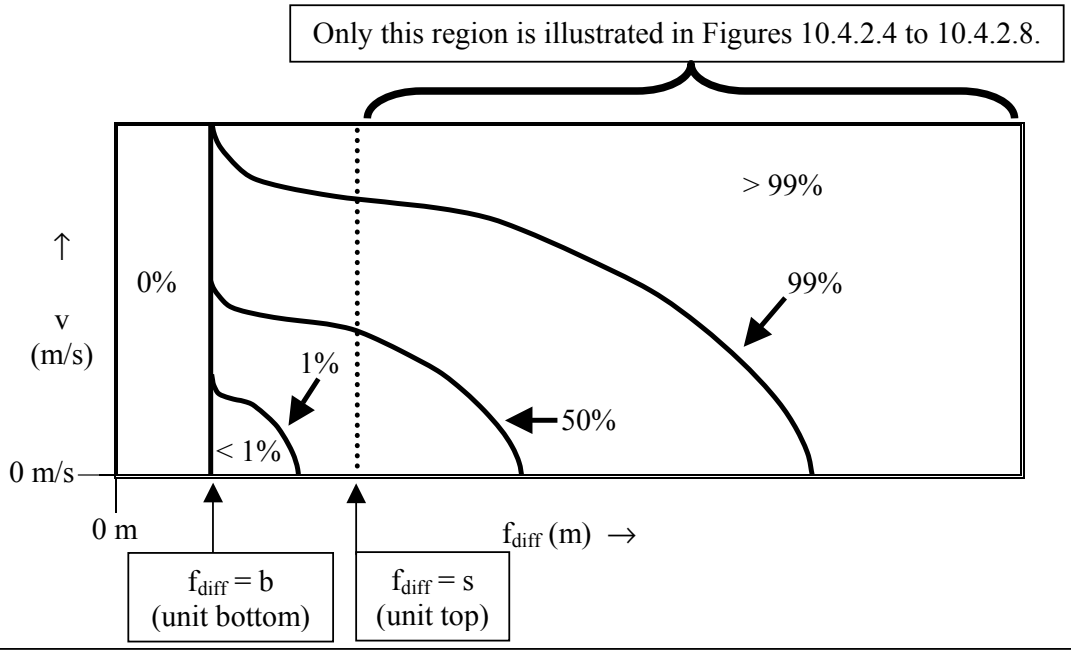


Figure 10.4.2.4: f_{diff}-v Graph for Unit F (1.00 m × 0.75 m × 4+4 mm thick)

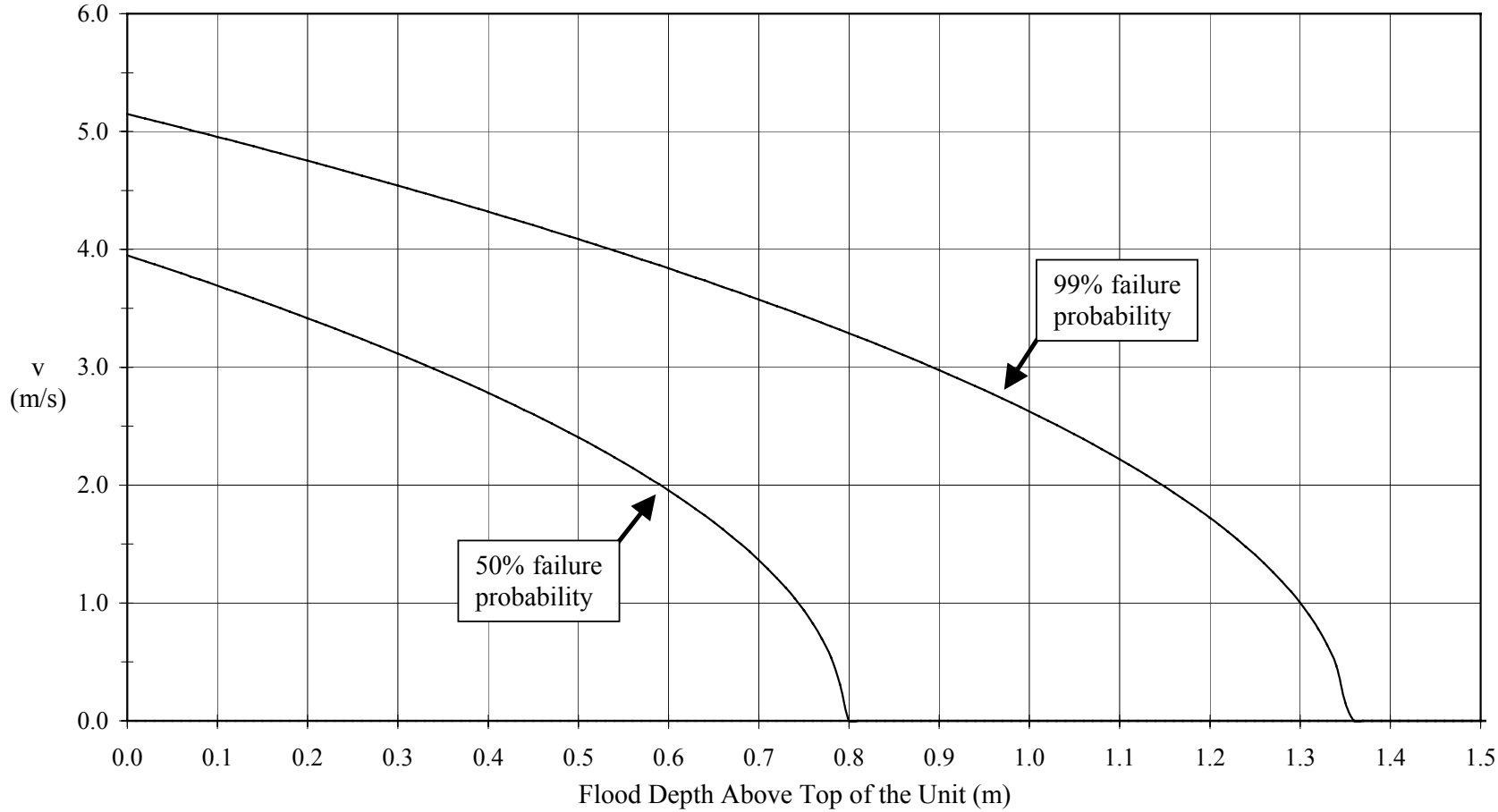


Figure 10.4.2.5: f_{diff}-v Graph for Unit G (1.00 m × 1.00 m × 4+4 mm thick)

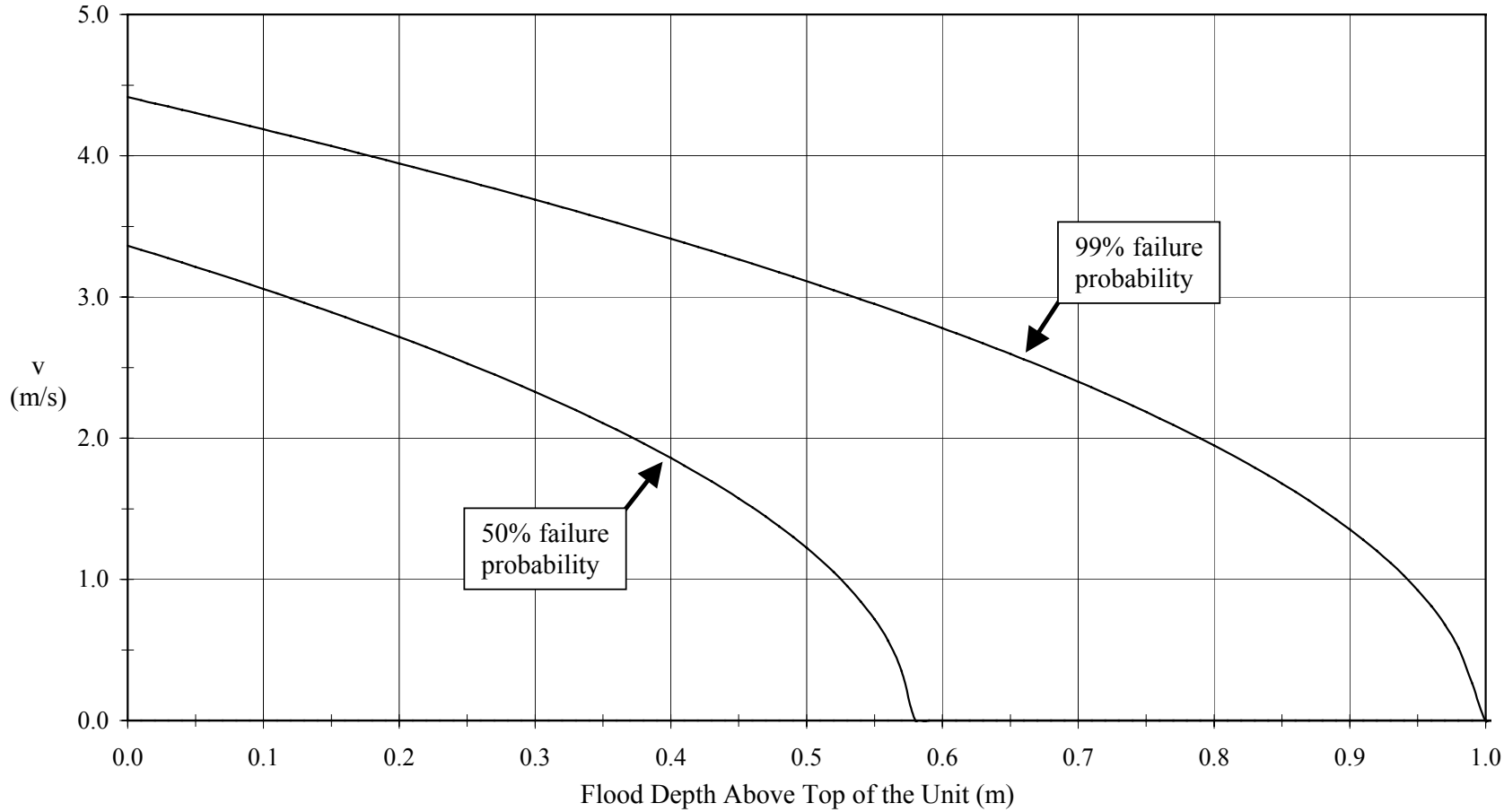


Figure 10.4.2.6: f_{diff}-v Graph for Unit K (0.75 m × 1.00 m × 4+4 mm thick)

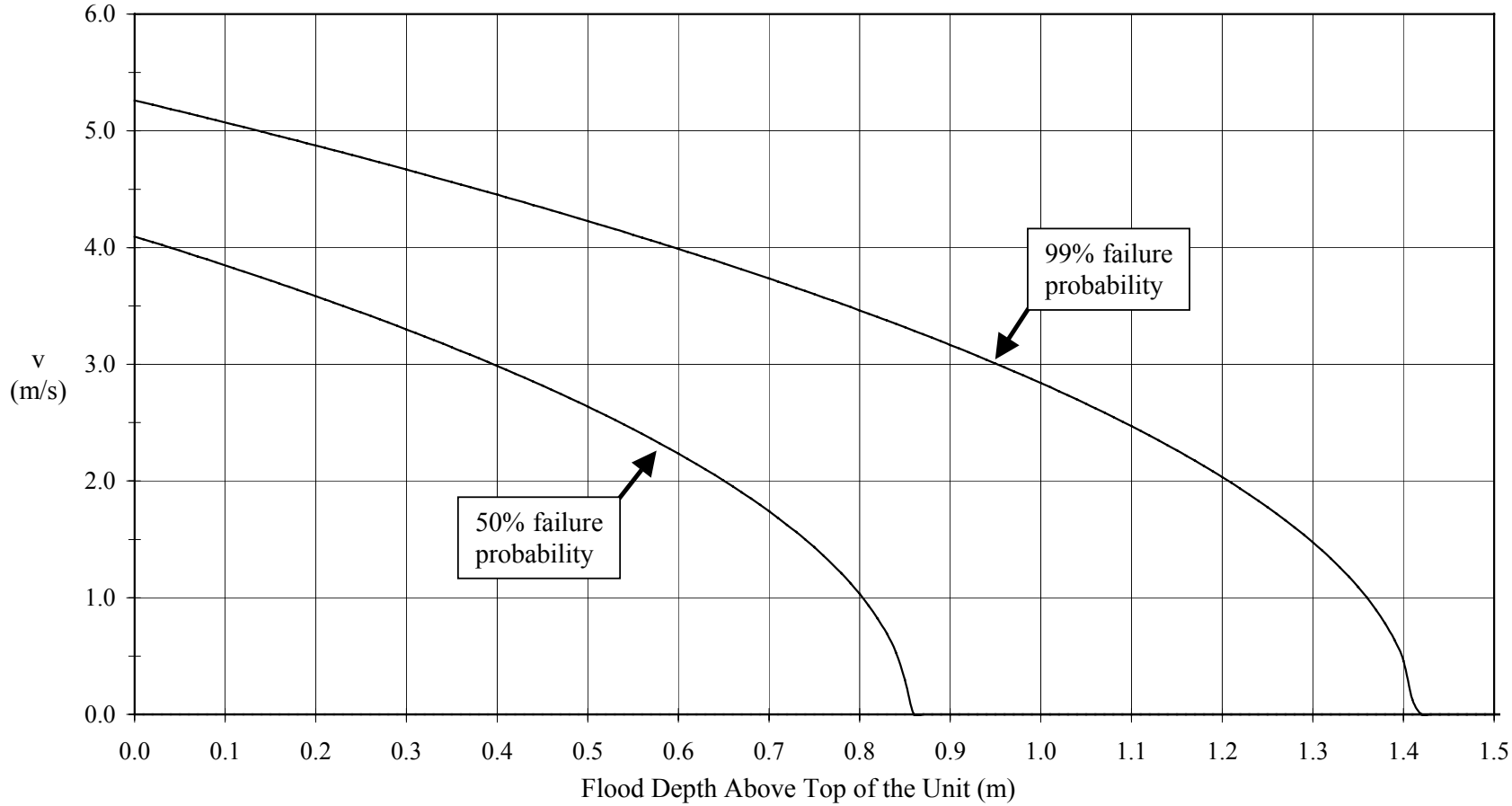


Figure 10.4.2.7: f_{diff}-v Graph for Unit Q (2.00 m × 1.00 m × 4+4 mm thick)

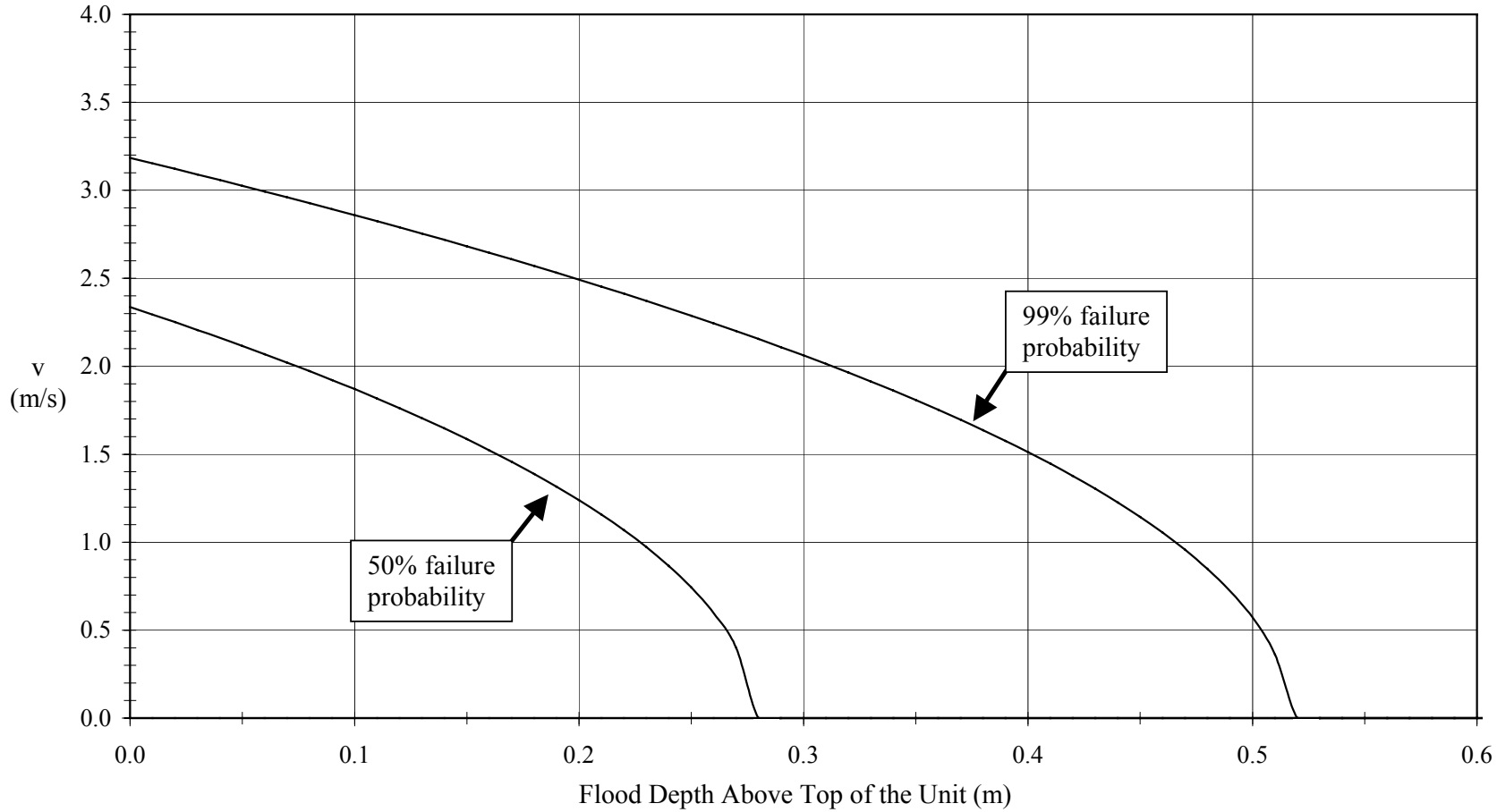
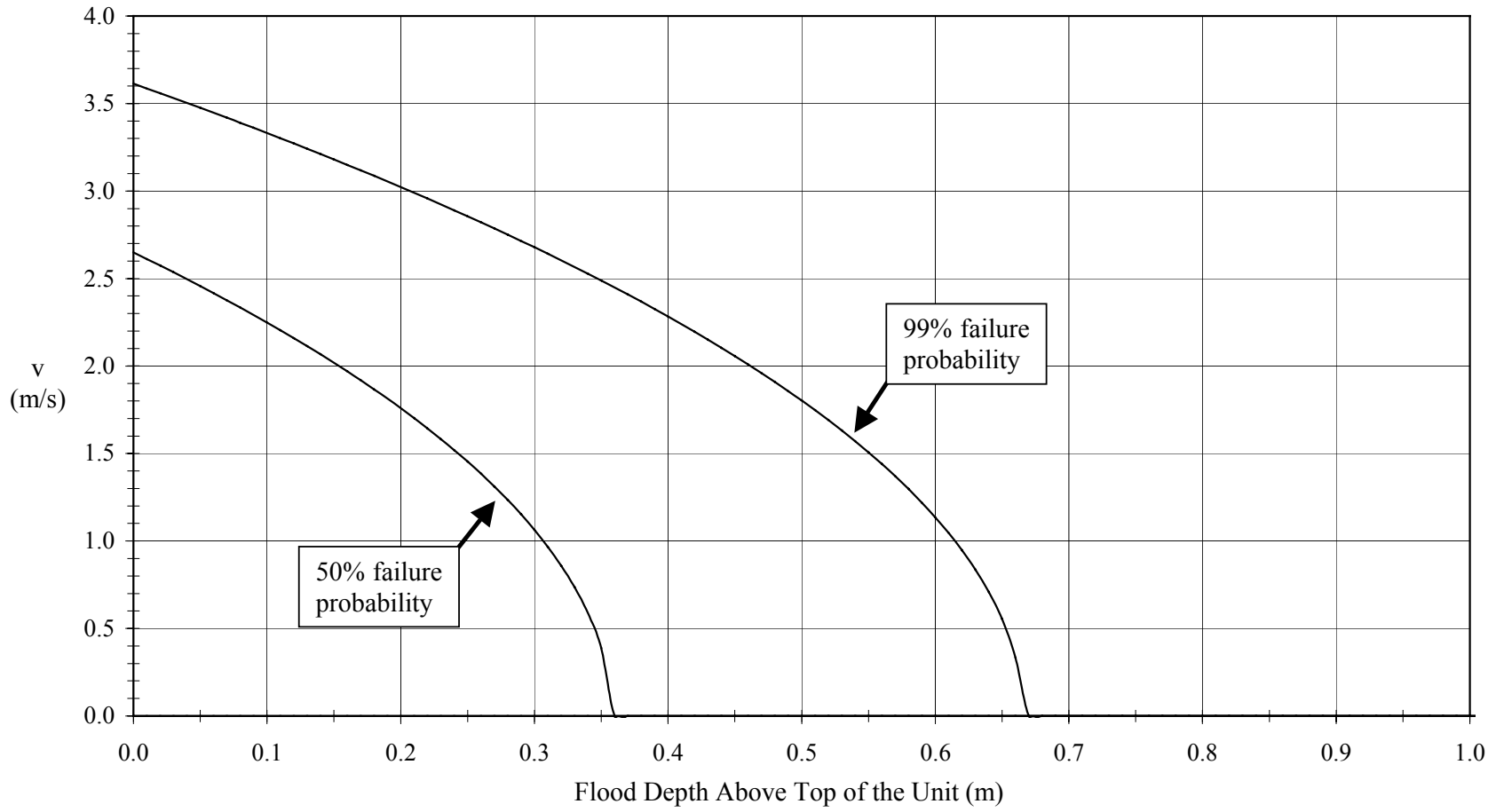


Figure 10.4.2.8: f_{diff}-v Graph for Unit R (2.00 m × 0.75 m × 4+4 mm thick)



From the data used to produce these graphs, a specific Wei(ξ) could theoretically be determined and then failure probabilities could be calculated precisely for given flood and unit parameters. This precision, however, would not reflect the accuracy, because large uncertainties would remain with selecting the specific Wei(ξ) and with carrying out these precise calculations. Overall, as many approximations and assumptions would be necessary to determine a specific Wei(ξ) as would be necessary to interpret the graphs using the three iso-σ_{max} curves. Precision would be gained by using a specific Wei(ξ) but accuracy would be neither lost nor gained through using or avoiding the specific Wei(ξ).

Table 10.4.2.1 illustrates the results used to approximately rank units by strength (see Section 10.5.1 for full discussion) through extracting key ΔP values from the f_{diff}-v graphs. The indicative value chosen is ΔP at each unit's centre, $\Delta P_{avg} = \Delta P_{y=b+(H/2)}$. $\Delta P_{p\%}$ represents ΔP for a failure probability of p%:

- $\Delta P_{1\%}$ is ΔP for σ_{max} = 20 MPa.
- $\Delta P_{50\%}$ is ΔP for σ_{max} = 60 MPa.
- $\Delta P_{99\%}$ is ΔP for σ_{max} = 80 MPa.

Corresponding values of velocity if f_{diff} = s ($v_{1\%}$, $v_{50\%}$, and $v_{99\%}$) and depth above the unit's top if v = 0 ($f_{diff} - s_{1\%}$, $f_{diff} - s_{50\%}$, and $f_{diff} - s_{99\%}$) are given too.

Table 10.4.2.1: Double-glazed Unit Failure Under Hydrostatic and Hydrodynamic Loads ($þ_{glass}$ = 4 mm. An approximate ranking of weakest to strongest panes is given; see Section 10.5.1 for discussion)

Unit	H (m)	W (m)	$\Delta P_{1\%}$ (kPa)	$f_{diff-s\ 1\%}$ (m)	$v_{1\%}$ (m/s)	$\Delta P_{50\%}$ (kPa)	$f_{diff-s\ 50\%}$ (m)	$v_{50\%}$ (m/s)	$\Delta P_{99\%}$ (kPa)	$f_{diff-s\ 99\%}$ (m)	$v_{99\%}$ (m/s)
Q	2.00	1.00	--	--	--	13	0.3	2.3	15	0.5	3.2
R	2.00	0.75	--	--	--	14	0.4	2.6	17	0.7	3.6
G	1.00	1.00	--	--	--	11	0.6	3.4	15	1.0	4.4
F	1.00	0.75	--	--	--	13	0.8	3.9	19	1.4	5.2
K	0.75	1.00	--	--	--	13	0.9	4.1	18	1.4	5.3
C	1.00	0.50	--	--	--	16	1.1	4.6	23	1.8	6.0
H	0.75	0.75	4	0.0	0.9	16	1.2	4.8	23	1.9	6.1
M	0.50	1.00	4	0.1	1.5	17	1.4	5.2	25	2.2	6.6
E	0.75	0.50	6	0.2	2.2	22	1.8	6.0	32	2.8	7.4
L	0.50	0.75	6	0.3	2.7	23	2.0	6.3	33	3.0	7.7
A	1.00	0.25	9	0.4	2.8	29	2.4	6.9	39	3.4	8.2
P	0.25	1.00	7	0.6	3.5	27	2.6	7.2	38	3.7	8.5
I	0.50	0.50	10	0.7	3.7	34	3.1	7.8	47	4.4	9.3
B	0.75	0.25	14	1.0	4.5	45	4.1	8.9	60	5.6	10.5
O	0.25	0.75	13	1.2	4.8	44	4.2	9.1	60	5.8	10.6
D	0.50	0.25	24	2.1	6.5	75	7.2	11.9	100	9.7	13.8
N	0.25	0.50	24	2.3	6.7	75	7.3	12.0	101	9.9	13.9
J	0.25	0.25	45	4.3	9.3	138	13.6	16.3	184	18.2	18.8

The principal trend is that failure ΔP increases as R decreases.

10.4.3 Sensitivity Analysis

Except for E, ν, and mapping Ω onto failure probabilities, the parameters for this analysis are defined with a high degree of accuracy and precision. Section 10.3.1 explained that variations in ν result in maximum errors of 3% (Aalami and Williams, 1975) which are not large enough to be of concern.

As long as E > 0, the choice of E has negligible influence. This consequence arises from the calculation method based on Aalami and Williams (1975). E is used in the calculations, but effectively cancels out. E cancels out completely when linear interpolation is used for sections of the graphs in Aalami and Williams (1975).

Mapping Ω onto failure probabilities is the biggest potential source of error. Since this procedure applies equally to all units, choices made would not influence analysis of the relative strength of different units. ΔP for failure would be affected. For changes of under 5% in each σ_{max} assigned to a failure probability, ΔP for that failure probability changes approximately linearly. For example, if σ_{max} = 63 MPa for $\Delta P_{50\%}$ rather than σ_{max} = 60 MPa, then the value of $\Delta P_{50\%}$ reported in Table 10.4.2.1 would be 5% too low.

Thus, the uncertainties in the method for mapping Ω onto failure probabilities could yield significant errors in absolute unit strength reported here. Extensive literature and expert judgement have been used to develop this method because other options with fewer uncertainties are not available. Determining whether or not this method is valid will be completed in Section 10.5 by comparing the results obtained here to past studies (Sections 10.1.2 and 10.1.5).

10.5 Discussion

10.5.1 Strength of Glass Panes

Tables 10.4.1.1 and 10.4.2.1 indicate that $\Delta P_{50\%}$ for the largest units is in the range 10-20 kPa. Smaller units have $\Delta P_{50\%}$ spread out, starting at 20 kPa and reaching 138 kPa for the smallest unit. The values for $\Delta P_{50\%}$ for specific units are almost exactly double the failure ΔP for many units of corresponding sizes reported in Section 10.1.2 from the literature review. This situation is expected because most studies in the literature review examined single-glazed units while the work reported in this Chapter examines double-glazed units. As noted in Section 10.2.4, double-glazed units are expected to have failure ΔP approximately double that of single-glazed units. Some units in Section 10.1.2 display high variance from the results found here, explicable by differences in β_{glass} and the wide range of ΔP at which a specific unit could fail due to the uncertainties involved. Overall, the $\Delta P_{50\%}$ range for double-glazed units is approximately double the range predicted for single-glazed units based on empirical evidence (Section 10.1.5) which somewhat validates the method and assumptions chosen.

R is the most significant factor in determining unit strength. As R increases, strength decreases.

One other important factor regarding the order proposed in Tables 10.4.1.1 and 10.4.2.1 is the error in the values. Given the errors associated with reading the graphs in Aalami and Williams (1975) along with the error from ν, an estimation of the errors for ΔP, f_{diff}, and v is less than ± 15% each. The clusters which appear within ± 15% of each other—considering ΔP, f_{diff}, and v—are units FGKQR, CHM, EL, AP, I, BO, DN, and J. No anomalies appear. Despite the asymmetry of the hydrostatic load, the unit's orientation for similar dimensions, i.e. the H/W ratio, does not appear to significantly affect unit strength.

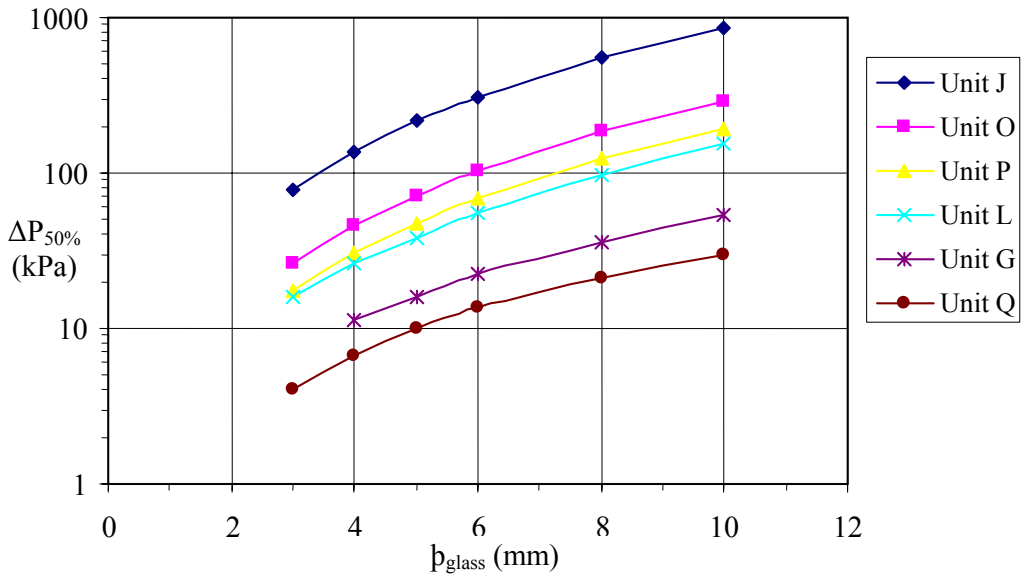
The ± 15% error is relatively small, but could change the order of the weakest panes represented by the FGKQR and CHM clusters. Changes in the order within the other clusters would not be as significant with respect to unit strength because the units within each cluster have the same dimensions, although different orientations.

Unit orientation may not manifest significantly with regard to unit strength but it does affect f_{diff} required for failure. For example, from Table 10.4.2.1, units P and A have similar strengths yet the 0.75 m difference in H becomes a 0.75 m difference in f_{diff} at failure. Determining the differences amongst units of f_{diff} at failure also entails analysing b. As discussed in Section 10.5.2, such analysis is not necessary.

10.5.2 Strengthening Glass Panes

As noted in Sections 10.2.4 and 10.5.1, double-glazed units are twice as strong as single-glazed units because the load is shared between the two panes. Using triple-glazed units would further increase the unit strength because the load would be shared between the three panes. The move towards using double-glazed units for energy efficiency is advantageous for decreasing the chance of glass failure in residences during floods but may have environmental and economic consequences.

Figure 10.5.2.1: Variation of Unit Strength with $þ_{glass}$ for Uniform Loads



$þ_{glass}$ has a strong influence on unit strength. A 2 mm increase in the thickness of each pane more than doubles the strength of most double-glazed units, but as R increases, $\partial\Delta P_{50\%} / \partial þ_{glass}$ decreases (Figure 10.5.2.1, drawn for uniform loads to avoid inconsistencies due to varying H). A change towards thicker glass is unlikely to occur legislatively, but any movement towards thinner glass should be resisted if the goal would be to prevent window and door units breaking due to flood actions.

H and W have an influence because strength decreases as R increases (Section 10.5.1). Demanding smaller units may not be desirable for aesthetics and could lead to other problems. Schwartz *et al.* (1989) identify design and installation factors which make small double-glazed units (dimensions approximately 0.29 m × 0.36 m × 2.3+2.3 mm) more susceptible to sudden fracture than larger double-glazed units. Without considering the full life cycle of the entire unit, assuming that smaller units are stronger could be erroneous for specific cases.

Efforts to strengthen glass must be placed in a wider context. Reasons other than protection from flood actions exist for strengthening window and door glass, such as protection from accidental impact and theft. Conversely, reasons exist for not strengthening window and door glass, such as providing a final chance to escape a fire. As well, trade-offs between safe breakage and non-breakage must be considered. For example, in the case of a child running into a door, safe breakage may absorb more energy than non-breakage thereby lessening the chance of severe injury.

The final context to consider for glass failure is that of wall panel failure (Chapter 9). Chapter 6 notes that more than 99% of window units with H,W > 0.25 m have b ≥ 0.80 m. Therefore, s ≥ 1.1 m, often above f_{diff} for wall failure. When H,W ≤ 0.25 m, comparing Table 10.4.2.1 with Chapter 9 shows that $\Delta P_{1\%}$ is higher than ΔP required to collapse most walls. Clearly, window units have a negligible failure probability before walls would collapse. Since wall failure causes more damage than glass failure (Table 1.4.1, the damage scale), considering b and s for window glass yields the result that windows are so high above the ground that window glass failure is not relevant for developing vulnerability matrices as defined in Chapter 3.

Window glass failure would be relevant for situations where f > 1 m but f_{diff} = 0 m because v could cause failure. Nonetheless, door units are lower and larger than window units, so in such a situation, the door glass would be expected to fail before the window glass. A scenario with $f_{diff} < b_{window\ glass}$ and waves would be another concern for window glass failure, although such wave scenarios would be likely to cause wall collapse too. Overall, window glass failure rarely enters the development of vulnerability matrices (Chapter 12).

Door glass is important. The cluster of large units FGKQR from Section 10.5.1 is representative of door units, many of which have b < 0.5 m. Smaller units do not need to be considered because either they are rare in doors or H,W ≤ 0.25 m which implies that the chance of glass failure is negligible before wall collapse. Therefore, only the cluster FGKQR in doors needs to be considered for glass failure and the reason for focusing on these units in previous sections is apparent.

10.5.3 Summary

The glass units important for the vulnerability analysis conducted for this study are large units on doors. Units FGKQR are the relevant sizes. The graphs to use for approximating the failure probability for a given f_{diff}-v combination are Figures 10.4.2.4 to 10.4.2.8. Despite this small range of results useful for this study, the analysis here produced many more results which could be useful for applications involving glass failure under circumstances where other possible failures are not important.

10.6 Critique

10.6.1 Data Available

All the required H/W ratios and boundary conditions are not provided in Aalami and Williams (1975). Other references on large deflection calculations are focused on computer modelling and tend to investigate specific cases rather than the comprehensive, generic approach offered by Aalami and Williams (1975). This gap in the literature is unfortunate although plenty information is available for making design choices (e.g. BS 6262 : 1982), even though the specific failure criteria may not be determinable. Irrespective, interpolation and extrapolation from Aalami and Williams (1975) is feasible to a high degree of confidence and was completed in this study. Aalami and Williams (1975) also use $\nu = 0.3$ which, as noted in Section 10.3.1, is not the ideal value. Although the discrepancy introduces small errors, providing graphs for various ν would increase accuracy.

The other major weakness in the literature is the lack of precedent for practical methods of mapping Ω onto failure probabilities. The studies in Section 10.1.4 are relatively theoretical and provide results which are not always directly applicable to the practical problem of predicting failure for in-service units. Similarly, studies describing failure ΔP (Section 10.1.2) or providing values for Ω (Section 10.3.1) do not relate these data to Wei(ξ).

This study, too, has proposed a method for calculating failure probability without defining a specific Wei(ξ). σ_{max} is mapped onto failure probabilities as a proxy for defining Ω. This method is justifiable and works adequately, yet still dodges the issue of linking the fundamental theory with the practice. Research into developing usable forms of Wei(ξ) based on parameters which are either available in the literature or measurable in the field, and preferably a combination, would be helpful to fully analyse the problem of units failing in residence windows and doors.

10.6.2 Special Glasses

Several types of special glasses exist but are rarely used in units for residential properties. Where they do occur, the unit's strength could be quite different from the results reported here. Creyke *et al.* (1982), for example, write that "The fracture strength of a piece of thermally strengthened glass may be 300 MN/m^2", up to an order of magnitude greater than

ordinary glass. LGIC (c. 2000a) notes that toughened glass is "about five times stronger than ordinary glass" whereas ISE (1999) states "Figures around 90 to 120 [MPa for Ω] or sometimes even higher are commonly quoted for toughened glass in the U.K.". Information from Doremus (1973) is summarised in Table 10.6.2.1.

Table 10.6.2.1: Strengths of Special Glasses (Doremus, 1973)

Glass Treatment	Approximate Maximum Strengthening Factor
Quench-hardening	6
Ion Exchange	10
Surface Crystallisation	17
Ion Exchange and Surface Crystallisation	22
Etching	30
Fire-Polishing	200
Second-Phase Particles	2

Using laminated glass does not appear to augment a unit's strength. LGIC (c. 2000b) says "Laminated glass is equivalent to float glass of the same total thickness in resisting uniform lateral wind loads". Vallabhan *et al.* (1993) compare laboratory work and mathematical modelling for laminated glass panes of dimensions 1.5 m x 1.5 m x 4.8 mm experiencing uniform lateral loads of up to 6.9 kPa. They conclude that differences emerge in the breaking behaviour rather than in the strength.

Laminated glass does not normally shatter as ordinary glass does because the layer between the glass sheets holds broken glass in place. Therefore, laminated glass behaves differently under applied pressures and does not produce dangerous shards due to failure, but it is not necessarily stronger. Whether or not laminated glass could withstand flood actions which break the glass would depend on the strength of the inner layer under sustained loads.

10.6.3 Temporal Factors

Glass strength varies with load duration. If the relative strength of glass is considered to be 1 for loads lasting 1 s, the relative strength becomes approximately 0.5 for loads lasting 1 year and approximately 1.7 for loads lasting 0.01 s (ISE, 1999). Most flood loads would be well within that time range, so the relative strength of glass is similar enough for any flood load. In particular, long-duration loads are unlikely considering that the rise rate of flood water inside a residence would normally be a few mm/s (Chapter 8).

Beason and Morgan (1984) propose "a simplified methodology" which involves multiplying σ_{max} by $(t/60)^{1/16}$, where t is the load's duration in seconds, in order to compare σ_{max} for differing durations. The range $1 \text{ s} \leq t \leq 3600 \text{ s}$ yields factors of between 0.77 and 1.3. Thus, to have a significant effect on the calculations here, the load durations would need to be very small or very large—outside the range of flood load durations. The factor of load duration would not significantly influence the calculations for flood loads.

The rate of change of ΔP may have an effect on glass strength. The consensus on the theory of glass failure is that glass may be assumed to be a linearly elastic material (Assumption 2 in Section 10.1.3). Thus, ΔP's rate of change would not affect the failure calculations, but may have an influence in reality. Finally, the in-service exposure conditions discussed in Section 10.6.4 must include the time for the conditions experienced.

10.6.4 In-service Exposure Conditions

Schwartz *et al.* (1989) note that "glass strength is significantly degraded by prolonged stress exposure". Their study concludes "the most significant loading factor appears to be temperature extreme, followed by changes in altitude of 305 m (1000 ft) or more, desiccant absorption, wind loading, and barometric pressure changes, in that order". For example, a sealed, double-glazed unit installed at an altitude different to that at which it was manufactured would experience expansion or compression of the air between the panes, placing continual stress on the glass. Quantifying the relative effects of each of these factors on different units throughout the case study sites would entail a long-term study. For this study, these factors may be assumed to insignificantly influence unit strength.

Beason and Morgan (1984), on the basis of four examples, conclude that "The strength of 20-yr-old, in-service glass plates is significantly less than the strength of new glass plates". They also state that "The strength of 20-yr-old, in-service glass plates tends to be relatively independent of the type of exposure".

The model used for glass failure is that failure occurs due to scratches or flaws on the surface (Section 10.3.1). The more scratches glass has, the more likely it is to fail. Units in residence doors and windows are exposed to an environment with plenty opportunity to obtain scratches, both indoors and outdoors. The units have a strong chance of being weakened with time, so older panes would be expected to be weaker than younger panes.

This correlation may not be useful because other factors must be considered. Schwartz *et al.* (1989) describe new, double-glazed units shattering: "Fractures originated at scratches in the glass along the edge of the glazing…one of the tradesmen at the job site reported that he had cleaned excess putty from the outside glass surface with razor knives after glazing installation". Orientation of the unit with respect to prevailing wind direction, wind breaks near the unit, or the presence of children or pets in the household could also have a greater influence than age.

Since these data were not within the mandate of the residence survey (Chapter 5), all units in residences must be treated as equal with respect to surface flaws. As noted above for Schwartz *et al.*'s (1989) factors, in-service exposure conditions cannot be taken into account explicitly. They were, however, considered implicitly because the choices for Ω and the matching failure probabilities (Section 10.3.1) were made based on a literature review (Table 10.3.1.3) which included studies considering panes in real situations (e.g. Bansal and Doremus, 1986; Makovička, 1998).

10.6.5 Conclusions

The main issues of concern regarding the glass failure calculations here are the interpolations and extrapolations due to lack of data availability (Section 10.6.1) and the influence of in-service exposure conditions on glass strength (Section 10.6.4). Despite these issues, Section 10.5 indicated that the results presented here match well with literature results.

10.7 Conclusions

Much research into glass failure has yet to be completed and caution is suggested in interpreting some of the results presented here. Irrespective, the method used here is validated by comparison with the literature and produces credible results. The reason is that the goal was to obtain only first-order results. Approximations were made, the calculations were conducted, and the results were reported in a manner that all uncertainties were encompassed. In particular, the interpretation of Ω was approximate and eliminated much information which could have been used, yet the method yielded practical, usable outcomes. The outcome is that only large door units are important for glass failure for the vulnerability

analysis conducted in this study. These units have a non-negligible probability of failing for b < f_{diff} < s and a high probability of failing for approximately $f_{diff} \geq 1.5$ m.

Many further results were obtained, yet are not reported because they are not relevant for solving the defined problem. Specifically, results for small units and single-glazed units are not used in this study. Overall, despite the lack of some information and the assumptions made, an adequate understanding and prediction of glass failure has been achieved for the units in residences which are most vulnerable to ΔP from f_{diff} and v. Nonetheless, reasons unrelated to flood actions must be considered for formulating policy on strengthening glass in residences.

11. Flood Failure Flowchart

11.1 Introduction

Chapter 8 indicated that water rises slowly enough inside residences that significant ΔP from f_{diff} would be imposed on a residence in the case of a fast-rise flood. Chapters 9 and 10 illustrate that failure in, respectively, cavity walls and glass with low b (the height above ground) and large R (area) would be expected frequently at $f_{diff} \approx 1.0$ m and in most circumstances for $f_{diff} \geq 1.5$ m.

Ambiguities in residence component failure present at the end of Chapter 7 may now be reconsidered. Failure in these components—roofs and non-glass components in doors and windows—is examined in this Chapter to produce a flood failure flowchart.

11.2 Developing the Flood Failure Flowchart

11.2.1 Roofs

Chapter 7 suggested that if a roof breaks in a flood, then the residence would be subjected to relatively significant flood actions which would be likely to cause failure in other residence components. Hence, complete destruction, DS5 (Table 1.4.1), could be assumed.

This assumption was confirmed in Chapter 9 which indicates that almost all residences subjected to f_{diff} which reaches the roof would be destroyed by wall collapse. Although roofs are weak compared to other residence components, roof failure does not need to be considered further.

11.2.2 Non-glass Components of Doors and Windows

As noted in Chapters 6 and 10, windows tend to have $b \geq 1.0$ m. Therefore, as with the glass in windows (Chapter 10), the non-glass components of windows are unlikely to experience ΔP sufficient to break them before walls collapse. Failure of window components does not need to be considered further. Ambiguities and lack of information regarding the relative strength of window components is no longer a cause for concern.

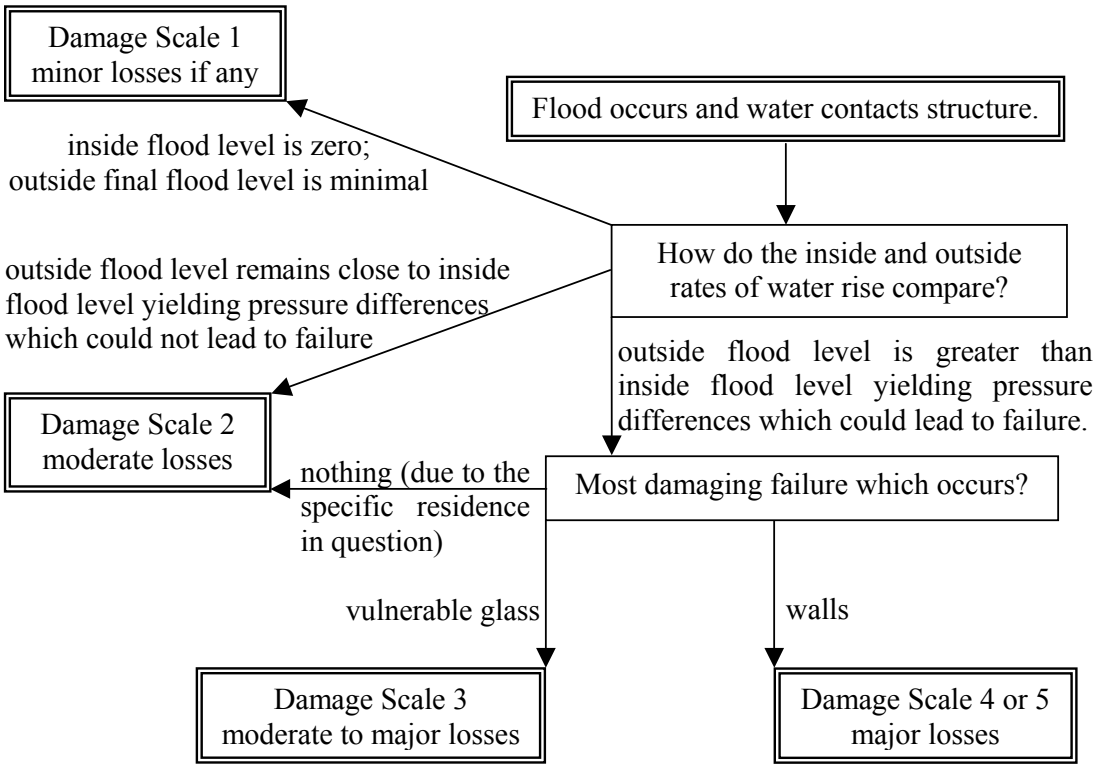
Door locks and some hinges also tend to have b ≥ 1.0 m, but some hinges will have b < 1.0 m. The door material and the door frame must be considered too. Blyth (1989) suggests that timber and plastic doors are strong but does not indicate a quantitative measure. Blyth (1989) also notes that plastic doors are "quite strong for residential use although not yet proven in the long term". Baird (2002) and McEvoy (2001-2) suggest that non-glass components of doors and windows would tend to withstand far greater ΔP than glass.

For doors without glass, walls fail at a low enough f_{diff} to suggest that wall failure is far more of a concern than door failure. This statement is particularly true since wall failure leads to DS4 or DS5 (Table 1.4.1) representing much worse damage that the DS3 encountered for door failure. Therefore, door failure does not need to be considered further.

11.2.3 Flood Failure Flowchart

The flowchart in Figure 11.2.3.1 illustrates the main failure pathways for a first-order analysis of residences in the case study sites subjected to ΔP from f_{diff} and v in floods. This schematic and the pathways represented are used to develop the vulnerability matrices (Chapter 12).

Figure 11.2.3.1: Flood Failure Flowchart



One concern with the analyses which helped produce Figure 11.2.3.1 is that situations (c) and (d) in Figure 3.2.2.1 may expose door and window components and roofs to flood actions which could cause them to fail. Residence components whose strength was not detailed due to their height above ground would need to be re-examined to ensure that they were indeed stronger than walls and glass experiencing similar pressure differentials. Chapter 3, though, scoped this study with the assumptions:

- The flood water reaches $f = f_{diff}$ and v instantaneously.
- No water had contacted the residence before that f_{diff}-v combination was reached.

Another concern is the assumption that residence components have been manufactured, installed, and maintained properly so that they comply with the relevant British Standards and they would be at that compliance level when the flood strikes. In Chapters 7 to 10, few decisions were made exclusively on the basis of British Standards. Expert judgement was also used. Finally, the residences are relatively young—92% were built after 1960—hence it is likely that they do comply with many British Standards. Nonetheless, reliance on the British Standards is a weakness in this analysis.

Finally, a residence may have narrow, and thus strong, wall panels without any glass in ground floor doors. Therefore, f_{diff} could reach relatively large values without breaking walls. Doors and windows, and possibly even roofs, could fail without the walls being damaged. Three-quarters of residences had low, large glass units in ground floor doors. At least one unambiguously wide wall panel on the ground floor could be identified in almost all residences. Therefore, residences with narrow wall panels and without any glass in ground floor doors are rare. Ignoring the possibility of such a residence still yields a first-order analysis of losses in the case study sites.

Despite these concerns, the assumptions permit the first-order analysis which is desired for this study. Given the assumptions made, caution is warranted in interpreting the results. Nonetheless, this study tackles the potential failure modes believed to be most significant for most observed residences to a level as best as feasible with the information currently available. Future studies would be appropriate for filling in gaps.

11.3 Conclusions

The main pathways by which flood water can enter and damage residences have been identified and prioritised. Uncertainties remain due to lack of information and weaknesses in the analysis. Neither literature nor experts were found which systematically identify or categorise potential failure modes in floods. Thus, prior recommendations on flood damage reduction did not necessarily have a strong basis. The flood failure flowchart (Figure 11.2.3.1) presented here provides initial, first-order insight into the main failure modes which should be, and which have been, quantified in detail.

12. Vulnerability Matrices

12.1 Introduction

Chapter 11 established that door glass breaking and walls collapsing were liable to be the main impacts of the flood actions of ΔP from f_{diff} and v on residences in the case study sites. Chapter 8 indicated that substantial ΔP could be experienced by glass and walls during floods. Chapters 9 and 10 quantified the f_{diff}-v combinations which would cause failure in glass and walls respectively.

This information now requires collation. As noted in Table 1.4.1, the damage scale, wall failure yields more damage than glass failure. The results from glass failure and wall failure must be merged to reflect this situation. In deciding that large, low door units would be of principal concern (Chapter 10), this process was started. The next stage is to formally combine the results to produce vulnerability matrices as previewed in Chapter 3.

12.2 Procedure

Chapter 6 illustrated that residences could be classified through the two dimensions of eight A-Π pairs and $j \in \{1,2,3,4\}$ where:

- A is residence plan area [m^2].
- j is the number of storeys or levels in a residence [no units].
- Π is external residence perimeter [m].

Thus, 32 different residence categories are considered. The obvious plan would be to fill in a vulnerability matrix, similar to Table 12.2.1 which is based on the discussion in Section 3.3.2, for each A-j-Π combination. As will be explained in this Section, Π was deemed to be of secondary importance. Only the twelve A-j combinations need to be considered. The explanation starts with Table 12.2.2 which indicates the importance of each of these variables for each damage scale level.

Table 12.2.1: Vulnerability Matrix for First-Order Vulnerability Profile

v_{max}	$f_{diff\,max}$ 0.0 m	0.5 m	1.0 m	1.5 m	2.0 m	[…]
0.0 m/s						
0.5 m/s						
1.0 m/s						
1.5 m/s						
2.0 m/s						
[…]						

Table 12.2.2: Damage Scale for Floods with Dependent Variables

DS	Damage Description	Most Important Residence Characteristics
0	No water contact with structure.	--
1	Water contacts outside of structure but does not enter.	--
2	Water infiltrates or external features are damaged or removed by water or debris.	flood rise rate $\Rightarrow$ A and Π
3	Water or debris penetrates through a closed or covered opening (probably by breaking the opening); for example, a window or a door.	glass failure $\Rightarrow \Pi$ as a proxy for the number of doors
4	Water or debris penetrates through a route not including an opening (structural integrity is attacked); for example, a wall or roof.	wall failure $\Rightarrow$ j mainly but also A
5	Structure is damaged beyond repair; for example, walls collapse, the structure moves, or the foundation is undermined.	wall failure $\Rightarrow$ j mainly but also A

The most severe damage levels, DS4 and DS5, depend on only A and j. As well, if only DS2 occurs—the minimum damage expected for most residences affected by floods—the specific FRR would make little difference to the damage outcome. The reason is that FRR values would be relatively low. The damage would occur due to water contact rather than due to physical forces from water rushing in quickly, as could occur at DS3, DS4, and DS5. Therefore, calculating the specific FRR is not needed for the vulnerability matrices. An indication that DS2 would occur suffices.

Next, the algorithm below was developed for producing each vulnerability matrix based on the template of Table 12.2.1:

1. All $f_{diff} = 0.0$ were labelled "DS0". The assumption that f_{diff} and v occur instantaneously implies that if $f_{diff} = 0$, then $f = 0$. Without flood water contacting the structure, damage normally does not occur.

2. f_{diff}-v combinations producing the smallest value of ΔP needed for failure of strong wall panels (W = 2.0 m, λ = 0.4) were identified. These f_{diff}-v combinations, and all combinations below and to the right in the vulnerability matrix, were labelled "DS5".
3. f_{diff}-v combinations producing the smallest value of ΔP needed for failure of weak wall panels (W = 5.0 m, λ = 0.0) were identified. These f_{diff}-v combinations were labelled "DS4 if weak wall panels".
4. All f_{diff}-v combinations between "DS4 if weak wall panels" and "DS5" were also labelled "DS4 if weak wall panels".
5. If an f_{diff}-v combination labelled "DS4 if weak wall panels" would cause failure in medium-strength wall panels (W = 2.0 m, λ = 0.0 or W = 5.0 m, λ = 0.4) then that f_{diff}-v combination was changed to "DS4".
6. All remaining f_{diff}-v combinations were labelled "DS2". Because "DS4 if weak wall panels" is a conditional statement, DS2 was added to these f_{diff}-v combinations.
7. All f_{diff}-v combinations containing "DS2" were checked for DS3 (see method below). A statement was added indicating that DS3 would be either likely or unlikely if vulnerable units were present. Each f_{diff}-v combination for which DS3 needed to be checked had $f_{diff} \leq 1.0$ m, confirming that only large door units with low b (height above ground of the door's bottom) needed to be examined.

DS3 was checked by first noting that almost all residences in Hull and approximately half the residences on Canvey Island have large door units with low b (Chapter 6). Therefore, DS3 is a possibility.

For a residence with several vulnerable units, the probability of none of them failing when $\Delta P_{avg} = \Delta P_{1\%}$ is $(1 - 0.01)^{\#\ units}$ where:

- ΔP_{avg} is the average pressure over the pane under consideration; and
- $\Delta P_{1\%}$ is the ΔP_{avg} needed for a 1% probability failure for glass.

No residence observed had more than ten vulnerable units. Even if this high number did occur, the probability of at least one failing at $\Delta P_{avg} = \Delta P_{1\%}$ would be less than 10%, a result which could be deemed "unlikely" that glass failure would occur. Thus, all residences are referred to by the statement that if $\Delta P_{avg} \leq \Delta P_{1\%}$ for the vulnerable door unit with the lowest b, then DS3 would be deemed to be "unlikely". Otherwise, DS3 is deemed to be "likely". This approach is conservative because failure would be deemed "likely" for any failure probability

over 10% and for many failure probabilities under 10%. The sensitivity of results to this choice is low because DS3 rarely appears in the vulnerability matrices.

Therefore, the number of doors for each residence does not affect the development of the vulnerability matrices. Using Π as a proxy for the number of doors, as indicated in Table 12.2.2, is not needed. Since Π is needed for neither DS2 (FRR is not being calculated exactly) nor DS3, then Π is not needed for creating the vulnerability matrices. Although the original intention was to produce 32 vulnerability matrices, one for each A-j-Π combination, developing the procedure has yielded the result that only the twelve A-j combinations are relevant. These vulnerability matrices appear in Section 12.3 as vulnerability profiles. f_{diff}-v combinations which yielded the same failure outcome are outlined in order to highlight f_{diff}-v zones of a specific failure type.

If FRR or exact failure probabilities for glass are needed, then Π must be considered. Alternatively, the specific characteristics of the residence in question, including A and j, could be used to create a vulnerability profile specific to a residence, a group of residences, or a particular residence design.

As noted for wall failure (Chapter 9) and glass failure (Chapter 10), the vulnerability matrices presented here do not factor in b, the distance from ground level to the ground floor of residences. This distance is 0.2-0.3 m for most residences in the case study sites, but would vary with location or specific residences considered. Before the vulnerability matrices may be applied, as illustrated in Chapter 13, b for residences must be taken into account.

12.3 Results: Vulnerability Profiles

Table 12.3.1: Vulnerability Profile for Residences with A = 38 m^2 and j = 1

Maximum Flood Velocity	Maximum Flood Depth Differential			
	0.0 m	0.5 m	1.0 m	1.5 m +
0.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS5
0.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS5	DS5
1.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS5	DS5
1.5 m/s	DS0	DS4	DS5	DS5
2.0 m/s	DS0	DS4	DS5	DS5
2.5 m/s	DS0	DS4	DS5	DS5
3.0 m/s	DS0	DS4	DS5	DS5
3.5 m/s	DS0	DS4	DS5	DS5
4.0 m/s	DS0	DS4	DS5	DS5
4.5 m/s +	DS0	DS5	DS5	DS5

Figure 12.3.1: Example from Canvey Island for Table 12.3.1

Table 12.3.2: Vulnerability Profile for Residences with A = 55 m^2 and j = 1

Maximum Flood Velocity	Maximum Flood Depth Differential			
	0.0 m	0.5 m	1.0 m	1.5 m +
0.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS5
0.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS5
1.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS5
1.5 m/s	DS0	DS4	DS4	DS5
2.0 m/s	DS0	DS4	DS5	DS5
2.5 m/s	DS0	DS4	DS5	DS5
3.0 m/s	DS0	DS4	DS5	DS5
3.5 m/s	DS0	DS4	DS5	DS5
4.0 m/s	DS0	DS4	DS5	DS5
4.5 m/s	DS0	DS4	DS5	DS5
5.0 m/s	DS0	DS4	DS5	DS5
5.5 m/s +	DS0	DS5	DS5	DS5

Figure 12.3.2: Example from Hull for Table 12.3.2

Table 12.3.3: Vulnerability Profile for Residences with A = 84 m^2 and j = 1

Maximum Flood Velocity	Maximum Flood Depth Differential			
	0.0 m	0.5 m	1.0 m	1.5 m +
0.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS5
0.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS5
1.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS5
1.5 m/s	DS0	DS4	DS4	DS5
2.0 m/s	DS0	DS4	DS4	DS5
2.5 m/s	DS0	DS4	DS4	DS5
3.0 m/s	DS0	DS4	DS5	DS5
3.5 m/s	DS0	DS4	DS5	DS5
4.0 m/s	DS0	DS4	DS5	DS5
4.5 m/s	DS0	DS4	DS5	DS5
5.0 m/s	DS0	DS4	DS5	DS5
5.5 m/s	DS0	DS4	DS5	DS5
6.0 m/s	DS0	DS4	DS5	DS5
6.5 m/s +	DS0	DS5	DS5	DS5

Figure 12.3.3: Example from Canvey Island for Table 12.3.3

Table 12.3.4: Vulnerability Profile for Residences with A = 38 m^2 and j = 2

Maximum Flood Velocity	Maximum Flood Depth Differential			
	0.0 m	0.5 m	1.0 m	1.5 m +
0.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS5
0.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS5
1.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS5
1.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS5
2.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS5
2.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS5
3.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS5	DS5
3.5 m/s	DS0	DS4	DS5	DS5
4.0 m/s	DS0	DS4	DS5	DS5
4.5 m/s	DS0	DS4	DS5	DS5
5.0 m/s	DS0	DS4	DS5	DS5
5.5 m/s	DS0	DS4	DS5	DS5
6.0 m/s	DS0	DS4	DS5	DS5
6.5 m/s +	DS0	DS5	DS5	DS5

Figure 12.3.4: Example from Canvey Island for Table 12.3.4 (Front and Back)

Table 12.3.5: Vulnerability Profile for Residences with A = 55 m^2 and j = 2

Maximum Flood Velocity	Maximum Flood Depth Differential				
	0.0 m	0.5 m	1.0 m	1.5 m	2.0 m +
0.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS5
0.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS5	DS5
1.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS5	DS5
1.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS5	DS5
2.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS5	DS5
2.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS5	DS5
3.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS5	DS5
3.5 m/s	DS0	DS4	DS4	DS5	DS5
4.0 m/s	DS0	DS4	DS5	DS5	DS5
4.5 m/s	DS0	DS4	DS5	DS5	DS5
5.0 m/s	DS0	DS4	DS5	DS5	DS5
5.5 m/s	DS0	DS4	DS5	DS5	DS5
6.0 m/s	DS0	DS4	DS5	DS5	DS5
6.5 m/s	DS0	DS4	DS5	DS5	DS5
7.0 m/s	DS0	DS4	DS5	DS5	DS5
7.5 m/s +	DS0	DS5	DS5	DS5	DS5

Figure 12.3.5: Example from Hull for Table 12.3.5 (Front and Back)

Table 12.3.6: Vulnerability Profile for Residences with A = 84 m^2 and j = 2

Maximum Flood Velocity	Maximum Flood Depth Differential				
	0.0 m	0.5 m	1.0 m	1.5 m	2.0 m +
0.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS5
0.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS5
1.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS5
1.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS5
2.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS5
2.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS5	DS5
3.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS5	DS5
3.5 m/s	DS0	DS4	DS4	DS5	DS5
4.0 m/s	DS0	DS4	DS4	DS5	DS5
4.5 m/s	DS0	DS4	DS4	DS5	DS5
5.0 m/s	DS0	DS4	DS5	DS5	DS5
5.5 m/s	DS0	DS4	DS5	DS5	DS5
6.0 m/s	DS0	DS4	DS5	DS5	DS5
6.5 m/s	DS0	DS4	DS5	DS5	DS5
7.0 m/s	DS0	DS4	DS5	DS5	DS5
7.5 m/s	DS0	DS4	DS5	DS5	DS5
8.0 m/s	DS0	DS4	DS5	DS5	DS5
8.5 m/s	DS0	DS4	DS5	DS5	DS5
9.0 m/s +	DS0	DS5	DS5	DS5	DS5

Figure 12.3.6: Example from Canvey Island for Table 12.3.6 (Front and Back)

Table 12.3.7: Vulnerability Profile for Residences with A = 38 m^2 and j = 3

Maximum Flood Velocity	Maximum Flood Depth Differential				
	0.0 m	0.5 m	1.0 m	1.5 m	2.0 m +
0.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS5
0.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS5
1.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS5
1.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS5	DS5
2.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS5	DS5
2.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS5	DS5
3.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS5	DS5
3.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS5	DS5
4.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS5	DS5	DS5
4.5 m/s	DS0	DS4	DS5	DS5	DS5
5.0 m/s	DS0	DS4	DS5	DS5	DS5
5.5 m/s	DS0	DS4	DS5	DS5	DS5
6.0 m/s	DS0	DS4	DS5	DS5	DS5
6.5 m/s	DS0	DS4	DS5	DS5	DS5
7.0 m/s	DS0	DS4	DS5	DS5	DS5
7.5 m/s	DS0	DS4	DS5	DS5	DS5
8.0 m/s +	DS0	DS5	DS5	DS5	DS5

Figure 12.3.7: Example from Hull for Table 12.3.7

Figure 12.3.8: Example from Hull for Table 12.3.8

Figure 12.3.9: Example from Canvey Island for Table 12.3.9 (Front and Back)

Table 12.3.8: Vulnerability Profile for Residences with A = 55 m^2 and j = 3

Maximum Flood Velocity	Maximum Flood Depth Differential				
	0.0 m	0.5 m	1.0 m	1.5 m	2.0 m +
0.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS5
0.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS5
1.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS5
1.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS5
2.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS5
2.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS5	DS5
3.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS5	DS5
3.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS5	DS5
4.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS5	DS5
4.5 m/s	DS0	DS4	DS4	DS5	DS5
5.0 m/s	DS0	DS4	DS5	DS5	DS5
5.5 m/s	DS0	DS4	DS5	DS5	DS5
6.0 m/s	DS0	DS4	DS5	DS5	DS5
6.5 m/s	DS0	DS4	DS5	DS5	DS5
7.0 m/s	DS0	DS4	DS5	DS5	DS5
7.5 m/s	DS0	DS4	DS5	DS5	DS5
8.0 m/s	DS0	DS4	DS5	DS5	DS5
8.5 m/s	DS0	DS4	DS5	DS5	DS5
9.0 m/s +	DS0	DS5	DS5	DS5	DS5

Table 12.3.9: Vulnerability Profile for Residences with A = 84 m^2 and j = 3

Maximum Flood Velocity	Maximum Flood Depth Differential					
	0.0 m	0.5 m	1.0 m	1.5 m	2.0 m	2.5 m +
0.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS4	DS5
0.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS4	DS5
1.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS4	DS5
1.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS5	DS5
2.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS5	DS5
2.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS5	DS5
3.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS5	DS5
3.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS4	DS5	DS5
4.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS5	DS5	DS5
4.5 m/s	DS0	DS4	DS4	DS5	DS5	DS5
5.0 m/s	DS0	DS4	DS4	DS5	DS5	DS5
5.5 m/s	DS0	DS4	DS4	DS5	DS5	DS5
6.0 m/s	DS0	DS4	DS5	DS5	DS5	DS5
6.5 m/s	DS0	DS4	DS5	DS5	DS5	DS5
7.0 m/s	DS0	DS4	DS5	DS5	DS5	DS5
7.5 m/s	DS0	DS4	DS5	DS5	DS5	DS5
8.0 m/s	DS0	DS4	DS5	DS5	DS5	DS5
8.5 m/s	DS0	DS4	DS5	DS5	DS5	DS5
9.0 m/s	DS0	DS4	DS5	DS5	DS5	DS5
9.5 m/s	DS0	DS4	DS5	DS5	DS5	DS5
10.0 m/s	DS0	DS4	DS5	DS5	DS5	DS5
10.5 m/s	DS0	DS4	DS5	DS5	DS5	DS5
11.0 m/s +	DS0	DS5	DS5	DS5	DS5	DS5

Table 12.3.10: Vulnerability Profile for Residences with A = 38 m^2 and j = 4

Maximum Flood Velocity	Maximum Flood Depth Differential				
	0.0 m	0.5 m	1.0 m	1.5 m	2.0 m +
0.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS5
0.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS5
1.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS5
1.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS5
2.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS5
2.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS5
3.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS5	DS5
3.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS5	DS5
4.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS5	DS5
4.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS5	DS5
5.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS5	DS5	DS5
5.5 m/s	DS0	DS4	DS5	DS5	DS5

6.0 m/s	DS0	DS4	DS5	DS5	DS5
6.5 m/s	DS0	DS4	DS5	DS5	DS5
7.0 m/s	DS0	DS4	DS5	DS5	DS5
7.5 m/s	DS0	DS4	DS5	DS5	DS5
8.0 m/s	DS0	DS4	DS5	DS5	DS5
8.5 m/s	DS0	DS4	DS5	DS5	DS5
9.0 m/s +	DS0	DS5	DS5	DS5	DS5

No examples of the residence in Table 12.3.10 were observed in the case study sites.

Table 12.3.11: Vulnerability Profile for Residences with A = 55 m^2 and j = 4

Maximum Flood Velocity	Maximum Flood Depth Differential					
	0.0 m	0.5 m	1.0 m	1.5 m	2.0 m	2.5 m +
0.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS4	DS5
0.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS4	DS5
1.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS5	DS5
1.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS5	DS5
2.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS5	DS5
2.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS5	DS5
3.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS5	DS5
3.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS5	DS5	DS5
4.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS5	DS5	DS5
4.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS5	DS5	DS5
5.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS5	DS5	DS5
5.5 m/s	DS0	DS4	DS4	DS5	DS5	DS5

6.0 m/s	DS0	DS4	DS5	DS5	DS5	DS5
6.5 m/s	DS0	DS4	DS5	DS5	DS5	DS5
7.0 m/s	DS0	DS4	DS5	DS5	DS5	DS5
7.5 m/s	DS0	DS4	DS5	DS5	DS5	DS5
8.0 m/s	DS0	DS4	DS5	DS5	DS5	DS5
8.5 m/s	DS0	DS4	DS5	DS5	DS5	DS5
9.0 m/s	DS0	DS4	DS5	DS5	DS5	DS5
9.5 m/s	DS0	DS4	DS5	DS5	DS5	DS5
10.0 m/s	DS0	DS4	DS5	DS5	DS5	DS5
10.5 m/s +	DS0	DS5	DS5	DS5	DS5	DS5

No examples of the residence in Table 12.3.11 were observed in the case study sites.

Table 12.3.12: Vulnerability Profile for Residences with A = 84 m^2 and j = 4

Maximum Flood Velocity	Maximum Flood Depth Differential					
	0.0 m	0.5 m	1.0 m	1.5 m	2.0 m	2.5 m +
0.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS4	DS5
0.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS4	DS5
1.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS4	DS5
1.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS4	DS5
2.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS4	DS5
2.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS4	DS5
3.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS5	DS5
3.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS5	DS5
4.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely	DS4	DS4	DS5	DS5
4.5 m/s	DS0	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS4	DS5	DS5
5.0 m/s	DS0	DS2 Even if glass doors, DS3 unlikely DS4 if weak wall panels	DS4	DS5	DS5	DS5
5.5 m/s	DS0	DS4	DS4	DS5	DS5	DS5

6.0 m/s	DS0	DS4	DS4	DS5	DS5	DS5
6.5 m/s	DS0	DS4	DS4	DS5	DS5	DS5
7.0 m/s	DS0	DS4	DS5	DS5	DS5	DS5
7.5 m/s	DS0	DS4	DS5	DS5	DS5	DS5
8.0 m/s	DS0	DS4	DS5	DS5	DS5	DS5
8.5 m/s	DS0	DS4	DS5	DS5	DS5	DS5
9.0 m/s	DS0	DS4	DS5	DS5	DS5	DS5
9.5 m/s	DS0	DS4	DS5	DS5	DS5	DS5
10.0 m/s	DS0	DS4	DS5	DS5	DS5	DS5
10.5 m/s	DS0	DS4	DS5	DS5	DS5	DS5
11.0 m/s	DS0	DS4	DS5	DS5	DS5	DS5
11.5 m/s	DS0	DS4	DS5	DS5	DS5	DS5
12.0 m/s	DS0	DS4	DS5	DS5	DS5	DS5
12.5 m/s +	DS0	DS5	DS5	DS5	DS5	DS5

Figure 12.3.12: Example from Hull for Table 12.3.12 (Front and Back)

12.4 Conclusions

A system to predict the consequences for a residence of given vulnerability characteristics, A and j, being subjected to specific hazard parameters, f_{diff} and v, has been provided. The form is vulnerability profiles as f_{diff}-v matrices. The system is resolved to 0.5 m for f_{diff} and 0.5 m/s for v. These relatively wide bands encompass many uncertainties described for each individual calculation which input into the creation of the vulnerability profiles.

One aspect yet to be factored in is b which leaves the vulnerability matrices as generic as possible. b must be considered before applying the matrices.

The vulnerability profiles presented here are for the residence types and categories developed through field surveys. The same tools could be applied to producing a vulnerability profile for a specific residence or for residences of other categories. Caution is warranted because such application must be for residences of a similar type to those observed, namely relatively modern residences in England constructed from unreinforced masonry.

Other cautions also emerge with respect to the initial, overall assumptions which were in effect throughout Part II and which were made to permit the development of the vulnerability profiles. These assumptions and the effect which they have on the results are described in Part III. Part III also discusses how, despite these assumptions, the system developed here is useful for better describing the risk environment of coastal, eastern England, as set out in Part I.

Part III

13. Analysis and Application

13.1 Introduction

The core research of this dissertation has been presented in Part II and collated into the vulnerability matrices (Chapter 12). These vulnerability matrices indicate the consequences for a residence of a particular type being subjected to a pressure differential (ΔP) from flood depth differential (f_{diff}) and flood velocity (v). The vulnerability matrices represent the outcome from solving the problem defined in Part I which has been attained through extending and applying knowledge to provide an original contribution to research.

This output may be further analysed and may be applied to formulating flood management recommendations. This Chapter explores the use and applicability of the vulnerability matrices along with the other analyses conducted in Parts I and II. Different approaches are tested and evaluated. The discussions are quantitative and qualitative and provide further original contributions to research, particularly geared at fulfilling the objectives outlined in Chapter 1.

13.2 Risk Quantification: Loss Equivalent Percentages (LEPs)

13.2.1 Defining Risk as a LEP Index

The vulnerability matrices (Chapter 12) provide a system for predicting the consequences for residences subjected to f_{diff}-v combinations. As explained in Chapter 2, vulnerability is one input into risk with the other necessary component being a description of the probability or frequency hazard, in this case f_{diff} and v. The risk equation involved summing (hazard parameter value exceedance probability) $\times$ (vulnerability to that hazard parameter value) over all values of the selected hazard parameter and over all residences. Hazard probability and vulnerability must be quantified to use this equation for risk calculations. The development of loss equivalent percentages (LEPs) illustrates one method.

The LEP method is developed here for a specific situation as an example of applying the vulnerability matrices for quantitative risk analysis. Assumptions made in this Section could

be refined, and sensitivities of results to those assumptions could be thoroughly checked, if the method were to be used for policy development or applied to an area other than the selected case study site of Canvey Island. For example, any area to which the vulnerability matrices are applied must have residence types similar to those found in Canvey Island and Hull. Otherwise, the same method could be followed, but the vulnerability matrices would need to be recalculated.

Chapter 2 defined vulnerability from UN DHA (1992) as the proportion of the total value of the residence lost due to the hazard experienced. To apply this definition, each cell of the vulnerability matrices must therefore be interpreted as a proportion of the total value of the residence lost. Since each cell is a damage scale (DS) level from Table 1.4.1 (repeated here), each DS must be assigned a LEP, the loss equivalent representing the percentage of the residence's value which is lost to a flood of given f_{diff} and v.

Table 1.4.1 (Repeated): Damage Scale for Floods

Damage Scale (DS) Level	Water Interaction with Structure
DS0	No water contact with structure.
DS1	Water contacts outside of structure but does not enter.
DS2	Water infiltrates (i.e. seeps or leaks in through small apertures). OR External features are damaged or removed by water or debris.
DS3	Water or debris penetrates through a closed or covered opening (probably by breaking the opening); for example, a window or a door.
DS4	Water or debris penetrates through a route not including an opening (structural integrity is attacked); for example, a wall or roof.
DS5	Structure is damaged beyond repair; for example, walls collapse, the structure moves, or the foundation is undermined.

Possibilities for calculating this LEP, which could be greater than one, are:

- Repair cost divided by market value.
- Repair cost divided by value to owner.
- Percentage of structure by floor area or volume damaged or lost.
- Repair cost divided by rebuilding cost (to check which option would be cheaper).
- Insurance payout divided by maximum possible payout (if a policy maximum exists).
- Insurance payout divided by premiums collected over customer's lifetime adjusted to present value.

LEP could refer to structure, contents, land, or a combination.

For practical, generic applications, LEP representing the percent market value of structure, contents, finishes, and systems lost by a residence in a flood disaster would be sought because it matches the UN DHA (1992) definition of vulnerability. Such figures assist in calculating an adequate estimate of economic loss at the level of individuals, communities, or the flood-affected area. This value is indicative of, rather than completely representing, the flood's impact on residential areas.

Despite the usefulness of this overall approach to providing an index which could be used as a basis for comparing risks, few studies provide data which could be used to calculate these flood LEPs for U.K. residences. The emphasis is on either a percentage of the sum insured (e.g. Black and Evans, 1999; Muir-Wood, 1999) or percent losses with 100% loss being defined on an arbitrary flood such as f = 3.0 m (e.g. Experian, 2000; N'Jai *et al.*, 1990). Non-U.K. studies, such as those in Table 2.4.2.1, yield similar concerns but nonetheless assist. Appelbaum (1985), for example, calculates damage as a percentage of the replacement value, but only as an intermediate step towards reaching relationships between depth and monetary damage. Davis and Skaggs (1992) and USACE (2000) present functions of percent damage of current value as a function of depth. Caution is needed in applying non-U.K. data to U.K. residences in case of differences in residence types and residence responses to flood actions. A gap remains in the availability of adequate data to calculate LEPs more comprehensively and for wider applications than considered before.

Therefore, for describing the LEP method in this study, illustrative values are estimated in Table 13.2.1.1 for a typical residence which would have no deliberate internal flood resistance.

Table 13.2.1.1: Illustrative LEPs for a Typical Residence

DS	0	1	2	3	4	5
Illustrative LEP	0%	2%	50%	60%	80%	100%

DS0 is 0% and DS5 is 100% by definition, although some items may be salvageable for DS5. For DS1, Experian's (2000) data state that total costs for a slow-rise flood just touching a residence (interpreted as $f \leq 0.0$ m) would be between 1.2% and 2.3% of the costs of a flood with f = 3.0 m. Graphs in Davis and Skaggs (1992) and USACE (2000) suggest that with $f \leq$ 0.0 m, residences without basements could suffer damage of up to 10% or 15% of structure

value and, separately, of content value, but the percentages are more often between 0% and 5%. 2% appears to be a reasonable, illustrative LEP for DS1.

For DS2, Black and Evans (1999) report that for f = 0.5 m, losses may be between 4% and 100% of the total insured value. As f increases, the lower threshold rises. Experian's (2000) data state that total costs for a slow-rise flood of 0.6 m would be between 40% and 60% of the costs of a flood with f = 3.0 m. This percentage rises and falls as f rises and falls respectively. Because water entry into an unprepared residence generally requires redecoration of all storeys affected, rewiring of the electrical system, likely damage to vehicles, and replacement of most contents including appliances and furniture, the resultant damage would be a significant proportion of the value of the entire residence. 50% appears to be a reasonable, illustrative LEP for DS2.

A LEP of 60% is an estimate for DS3 because no data were found. Doors or windows breaking would increase losses since broken components would need to be replaced and the rapid water entry could cause extra damage. A LEP of 80% for DS4 is also an estimate due to lack of data. Collapsed walls are expensive to repair and the broken masonry and force of water entry could cause damage. The result would still be less than the catastrophe of complete destruction (DS5). Therefore, the LEP for DS4 is placed halfway between the LEPs for DS3 and DS5.

For comparison, a residence deliberately constructed or retrofitted with high internal flood resistance is examined with illustrative LEPs estimated in Table 13.2.1.2. Actions to increase the internal flood resistance of a residence include raising the residence's electricity sockets, telephone points, cabling, and meters; removing ground floor carpets and valuables; and increasing the resistance and resilience of the materials and finishes used on the ground floor (more detail and discussion is given in Section 13.4).

Table 13.2.1.2: Illustrative LEPs for a Residence with High Internal Flood Resistance

DS	0	1	2	3	4	5
Illustrative LEP	0%	2%	10%	20%	70%	100%

DS0 is 0% and DS5 is 100% by definition, although some items may be salvageable for DS5. The focus in Table 13.2.1.2 is on high internal flood resistance, so damage at DS1 from water outside the residence should be similar to that experienced by a typical residence.

The remaining LEPs in Table 13.2.1.2 are estimates. Documents on increasing residence flood resistance generally indicate that increasing flood resistance will significantly reduce damage but quantitative estimates tend to be absent (e.g. BRESL, 1996; DTLR, 2002; EA/CIRIA, 2001b) due to the dependency on residence type and on the specific measures implemented and maintained. The LEPs for DS2 and DS3 aim to estimate the maximum level which damage should reach if flood resistance techniques are implemented and continued properly.

For DS4, flood resistance studies tend to have negligible comment on minimising damage if structural failure occurs, but wall reconstruction would be expected to be expensive. Damage from broken masonry and the force of water entering through the gap is liable to be significant too. Thus, the consequences for a residence with high internal flood resistance are likely to be similar to a typical residence. The essential act in increasing flood resistance of moving contents to a higher level or to another location may mitigate losses somewhat. The LEP for DS4 is set to 70%.

The values in Tables 13.2.1.1 and Table 13.2.1.2 are further critiqued in Section 13.3.2. Once the DS-to-LEP conversions are set, the distribution of the residence classes throughout the case study sites is needed. This distribution is known from Chapter 6. The distribution involving houses and flat columns, rather than flats, must be used in this instance because the number of storeys in the structure rather than the flat's level determines which vulnerability matrix to use. This approach introduces an inconsistency into the definition of "residence". The LEP for flat columns reflects the loss of the entire flat column rather than the loss of a particular flat or of the complete flat block. This simplification is appropriate for illustrating the LEP method by reducing the number of calculation steps.

The distribution of f_{diff}-v combinations in a given flood is also needed and is calculable from hydrodynamic models (Brown, 2003). With these data, a LEP index representing quantitative risk may be calculated. The LEP index represents the total value of all residences in a case study site expected to be damaged over a given timeframe due to f_{diff}-v combinations from potential floods.

Using this outline, this method of calculating risk as a LEP index will be demonstrated for a sea defence breach on Canvey Island due to a storm surge. Brown (2003) explains why hazard probabilities were not calculated, but he used hydrodynamic models to define a flood

and to provide the percent area covered by each f_{diff}-v combination (Section 13.2.2). Because Brown's (2003) models do not calculate hazard probability, the timeframe is effectively a single event with an occurrence probability of 100%.

The necessary assumptions for integrating the results from the hydrodynamic modelling and the vulnerability matrices are:

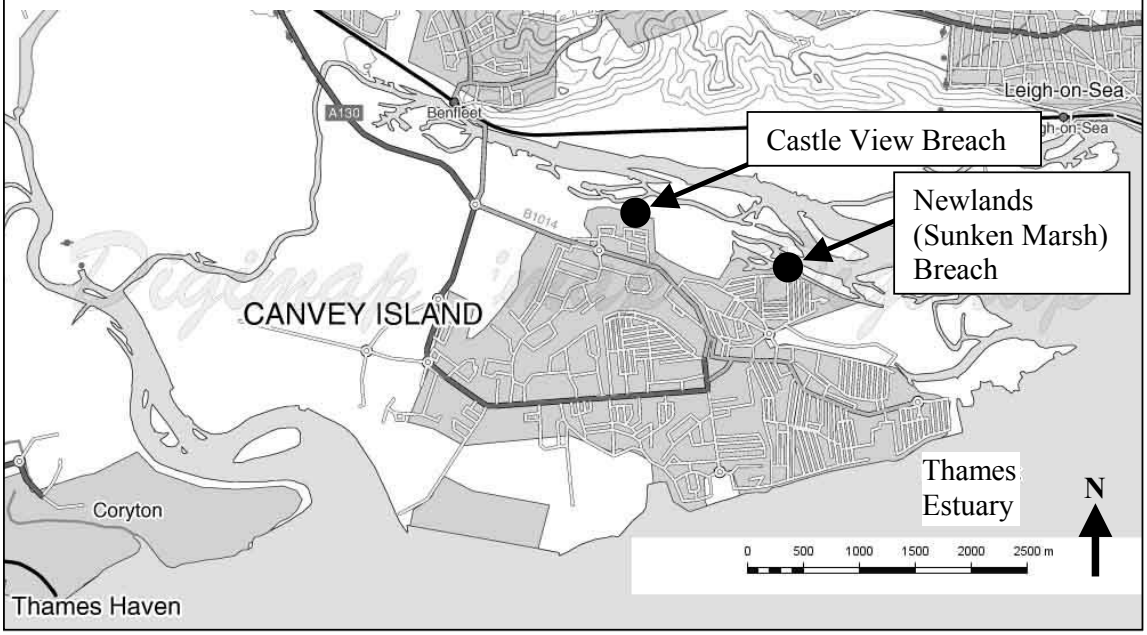
- Only horizontal velocities from the flood are considered. The effects of vertical velocities, suction, eddies, waves, and turbulence may be significant but could not be incorporated.
- All flood water impacts perpendicular to a residence.
- The effect of any residence collapses on the flood is not integrated into the model. If a line of houses or a flat block collapses, the characteristics of the hydrodynamic flood wave would be affected locally, but might not affect the ultimate outcome of the flood water across the case study site (Brown, 2003). If collapses were to significantly increase the bottom friction coefficient, then the ultimate outcome of the flood wave could be affected (Brown, 2003).
- Homogeneous distribution of residence types across the flooded area, in this case Canvey Island, was assumed. The field surveys indicated that this assumption appears to be adequately realistic, but it may not always be the case. Furthermore, this assumption would be unlikely to be robust through time because new development would not necessarily match the current resident types. Canvey Island has a high rate of new development with observations indicating that larger residences were generally replacing smaller ones. Irrespective, this assumption is made principally to illustrate the LEP method and an indication of how to avoid this assumption is provided in Section 13.2.6.
- The maximum velocity at a point in space occurs at the same time as the maximum depth differential at that point in space.
- The flood water reaches $f_{diff} = f_{diff\ max}$ and $v = v_{max}$ instantaneously.

These assumptions tend towards investigating a worst-case scenario with the exception of the assumption of homogeneous residence distribution which tends towards an average scenario.

13.2.2 Canvey Island Sea Defence Breach Scenarios: Hazard

Only the residential part of Canvey Island was considered in line with this dissertation's objectives (Section 1.3). This area is contiguous and totals 6.6 km^2. Based on an inspection

Figure 13.2.2.1: Locations of Simulated Sea Defence Breaches on Canvey Island (Reproduced from the 2001 Ordnance Survey map with the permission of the Controller of Her Majesty's Stationery Office, © Crown Copyright NC/01/26576.)



of the Canvey Island sea defences and an examination of the breach locations during the 1953 storm surge, Brown (2003) decided that the locations in Figure 13.2.2.1 would be most appropriate for simulating breaches. Both these locations are in the north of Canvey Island and are named after the adjacent communities: Castle View further west and Newlands, originally Sunken Marsh, further east.

The flood inundation over a period of 3 hours and 36 minutes from the time of the breach was examined in each scenario. Brown's (2003) investigations led him to assume that each breach would be 2.0 m high, representing instantaneous collapse of the exposed part of the concrete sea defences surrounding Canvey Island. The rest of the concrete wall, below this exposed part, is supported by earthen embankments. The assumption is that no further damage is sustained by the wall or the earthen embankment behind it. Brown (2003) also felt that two reasonable breach widths to consider for each location would be 50 m and 200 m. Such assumptions were necessary because of the high uncertainty and research gap in understanding how structural defences respond and potentially fail in flood situations (e.g. Sayers *et al.*, 2002).

The output of the hydrodynamic models is in the form of matrices providing the percent of the entire residential area of the case study site which experiences a certain f_{diff}-v combination (Tables 13.2.2.1 to 13.2.2.5). These matrices are presented for each scenario. Chapter 3

explains that Brown's (2003) model produces highly precise values for f, f_{diff}, and v but the smallest intervals which would be appropriate for developing vulnerability matrices would be approximately 0.5 m for f_{diff} and 0.5 m/s for v. Tables 13.2.2.1 to 13.2.2.5 therefore present ranges to be mapped onto the 0.5 value which falls within each range; e.g., 0.0-0.25 is mapped onto 0.0 and 1.25-1.75 is mapped onto 1.5.

The values presented in Tables 13.2.2.1 to 13.2.2.5 have been adjusted to reflect b, the distance between ground level and the bottom of the ground floor of residences. Chapter 6 noted that b = 0.3 m for Canvey Island. Therefore, the f_{diff} values been lowered 0.3 m from the models' output before calculating the percent area covered by each range. Subsequently negative values of f_{diff} were set to 0.0 m to ensure that these residences were contacted by the flood water. The choice between DS1 and DS2 for such residences was not straightforward. DS2 was chosen in this instance as a conservative assumption and to better reflect the lack of flood awareness and preparedness observed on Canvey Island.

A 50 m simulated breach in Castle View yielded a total flooded residential area of 0.79 km^2 representing 12% of the residential area of Canvey Island (Table 13.2.2.1).

Table 13.2.2.1: Percent of Residential Canvey Island Experiencing Each f_{diff}-v Combination for a 50 m Breach in Castle View (Total is 12%)

		f_{diff} – 0.3 (m)						
		0.00-0.25	0.25-0.75	0.75-1.25	1.25-1.75	1.75-2.25	2.25-2.75	2.75-3.25
v (m/s)	0.00-0.25	0.1	3.4	1.3	0.3	< 0.01	< 0.01	< 0.01
	0.25-0.75	0.1	2.8	1.6	0.6	0.8	0.1	< 0.01
	0.75-1.25	< 0.01	0.2	0.1	0.1	0.2	0.1	< 0.01
	1.25-1.75	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01
	1.75-2.25	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01
	2.25-2.75	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01

A 200 m simulated breach in Castle View yielded a total flooded residential area of 3.9 km^2 representing 59% of the residential area of Canvey Island (Table 13.2.2.2).

Table 13.2.2.2: Percent of Residential Canvey Island Experiencing Each f_{diff}-v Combination for a 200 m Breach in Castle View (Total is 59%)

		f_{diff} – 0.3 (m)							
		0.00-0.25	0.25-0.75	0.75-1.25	1.25-1.75	1.75-2.25	2.25-2.75	2.75-3.25	3.25-3.75
v (m/s)	0.00-0.25	3.5	6.6	3.1	0.3	< 0.01	< 0.01	< 0.01	< 0.01
	0.25-0.75	2.6	17.2	16.1	2.4	0.5	0.3	< 0.01	< 0.01
	0.75-1.25	0.2	1.3	2.3	0.8	0.4	0.7	0.1	< 0.01
	1.25-1.75	< 0.01	0.1	0.2	0.2	0.1	0.1	< 0.01	< 0.01
	1.75-2.25	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01
	2.25-2.75	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01
	2.75-3.25	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01

A 50 m simulated breach in Newlands yielded a total flooded residential area of 1.4 km^2 representing 21% of the residential area of Canvey Island (Table 13.2.2.3).

Table 13.2.2.3: Percent of Residential Canvey Island Experiencing Each f_{diff}-v Combination for a 50 m Breach in Newlands (Total is 21%)

		f_{diff} – 0.3 (m)						
		0.00-0.25	0.25-0.75	0.75-1.25	1.25-1.75	1.75-2.25	2.25-2.75	2.75-3.25
v (m/s)	0.00-0.25	3.9	4.4	0.8	< 0.01	< 0.01	< 0.01	< 0.01
	0.25-0.75	1.3	6.8	2.9	0.1	< 0.01	< 0.01	< 0.01
	0.75-1.25	0.1	0.3	0.2	< 0.01	< 0.01	< 0.01	< 0.01
	1.25-1.75	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01
	1.75-2.25	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01
	2.25-2.75	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01
	2.75-3.25	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01
	3.25-3.75	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01
	3.75-4.25	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01

A 200 m simulated breach in Newlands yielded a total flooded residential area of 4.8 km^2 representing 73% of the residential area of Canvey Island (Table 13.2.2.4).

Table 13.2.2.4: Percent of Residential Canvey Island Experiencing Each f_{diff}-v Combination for a 200 m Breach in Newlands (Total is 73%)

		$f_{diff} - 0.3$ (m)								
		0.00-0.25	0.25-0.75	0.75-1.25	1.25-1.75	1.75-2.25	2.25-2.75	2.75-3.25	3.25-3.75	3.75-4.25
v (m/s)	0.00-0.25	5.8	13.0	1.9	0.8	0.2	< 0.01	< 0.01	< 0.01	< 0.01
	0.25-0.75	3.1	22.3	9.1	5.7	3.1	0.6	< 0.01	< 0.01	< 0.01
	0.75-1.25	0.1	0.8	1.3	1.7	1.7	0.5	0.1	< 0.01	< 0.01
	1.25-1.75	< 0.01	0.1	0.1	0.3	0.2	0.1	< 0.01	< 0.01	< 0.01
	1.75-2.25	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01
	2.25-2.75	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01
	2.75-3.25	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01
	3.25-3.75	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01

The final simulated breach also examined a 200 m breach in Newlands, but the residences within 100 m of the sea wall where the breach occurs were removed from the model and replaced with thick vegetation (Table 13.2.2.5). This situation was simulated by changing the model's roughness coefficient in this area (Brown, 2003). This scenario effectively introduces a buffer zone where residences would not be constructed and which was not considered part of the residential area of Canvey Island. The total flooded residential area was 2.7 km^2 representing 41% of the residential area of Canvey Island.

Table 13.2.2.5: Percent of Residential Canvey Island Experiencing Each f_{diff}-v Combination for a 200 m Breach in Newlands with a 100 m Buffer Zone (Total is 41%)

		$f_{diff} - 0.3$ (m)								
		0.00-0.25	0.25-0.75	0.75-1.25	1.25-1.75	1.75-2.25	2.25-2.75	2.75-3.25	3.25-3.75	3.75-4.25
v (m/s)	0.00-0.25	10.1	7.9	0.9	0.2	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01
	0.25-0.75	3.0	9.8	4.2	1.6	0.5	< 0.01	< 0.01	< 0.01	< 0.01
	0.75-1.25	0.1	0.5	0.8	0.7	0.4	< 0.01	< 0.01	< 0.01	< 0.01
	1.25-1.75	< 0.01	< 0.01	0.1	0.1	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01
	1.75-2.25	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01
	2.25-2.75	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01

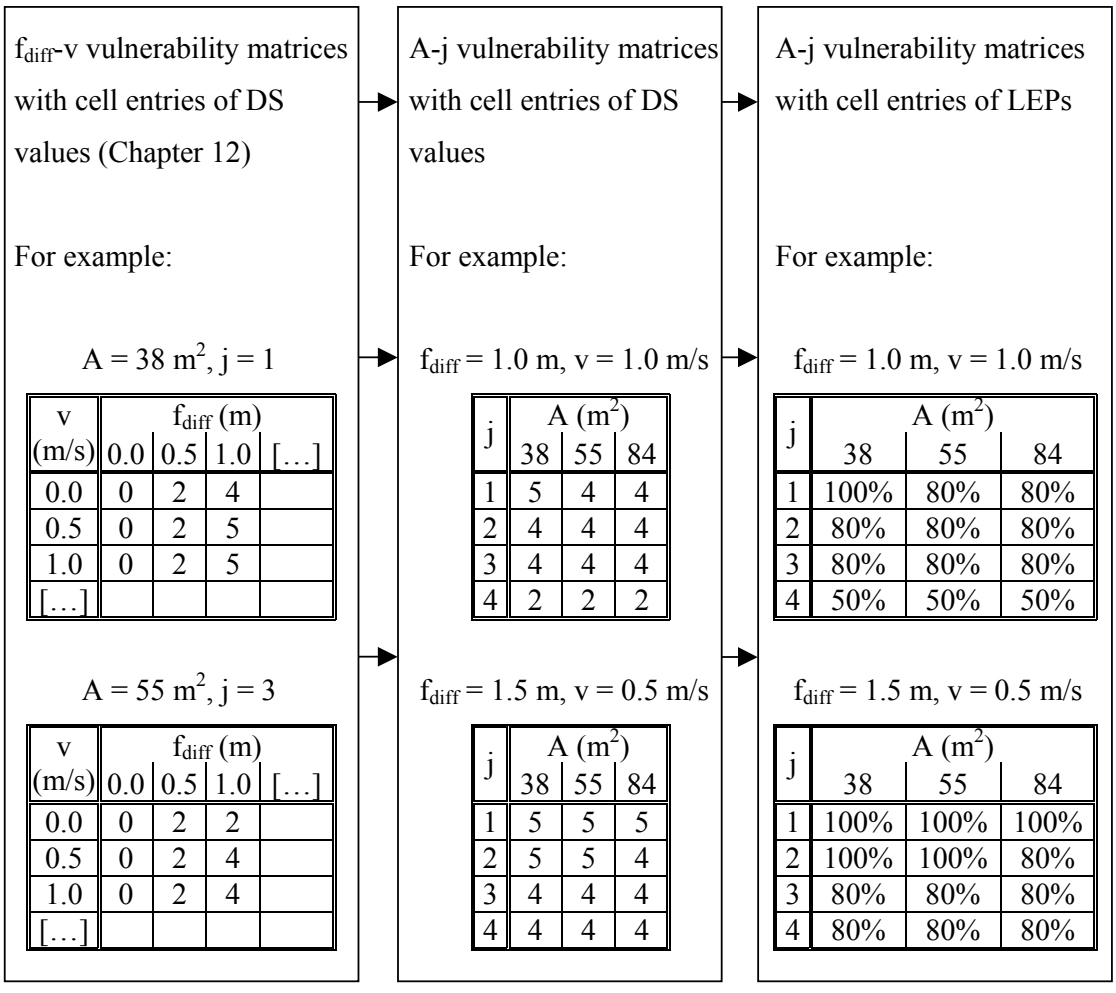
To keep the method described here as generic as possible, in the following sections f_{diff} generally refers to $f_{diff} - b$ which in this case is $f_{diff} - 0.3$ m. The reason is that b would vary from community to community. As long as the procedure here is followed step-by-step and

all the cautions that b must be considered are taken, then it is clearer to refer to only f_{diff} rather than f_{diff} – 0.3 m each time.

13.2.3 Canvey Island Sea Defence Breach Scenarios: Vulnerability and Risk

The next step in the LEP method involves manipulation of the vulnerability matrices from Chapter 12, which is done for ease of calculation and explanation. Each vulnerability matrix was developed for an A-j combination with a DS level provided for each f_{diff}-v combination. These matrices may be reconstructed so that each f_{diff}-v combination has a matrix which provides a DS level for each A-j combination (Figure 13.2.3.1). These DS levels are then converted into LEPs using Table 13.2.1.1 for illustration.

Figure 13.2.3.1: Reconstructing the Vulnerability Matrices



Some DS2 entries in the vulnerability matrices could be DS3 if large, low door glass exists and fails or could be DS4 if the residence has weak wall panels. The matrices in Chapter 12 display the specific cells where this situation arises. These two potential variations were built into the calculations by permitting a percent of each residence class to have glass doors which fail, weak wall panels, or both. For the explanation here, it is assumed that residences have neither glass doors which fail nor weak wall panels.

In the third column of Figure 13.2.3.1, the LEP entries are designated $LEP(A,j,f_{diff},v)$ to indicate that the LEP is specific to an A-j-f_{diff}-v combination. Similarly:

- The percent of each residence class on Canvey Island is designated $RC(A,j)$.
- The percent of residential area which experiences each f_{diff}-v combination for a given flood scenario is designated $h(f_{diff},v)$.

The illustrative assumption that the residences across Canvey Island are homogenous, as discussed in Section 13.2.1, is now enacted. With this assumption, for each f_{diff}-v, the following product is calculated:

$$\sum_{\text{All A,j for a specific } f_{diff},v} LEP(A,j,f_{diff},v) \times RC(A,j) = LEP(f_{diff},v)$$

$LEP(f_{diff},v)$ represents the LEP across Canvey Island assuming that the entire residential area is inundated by a single f_{diff}-v combination (Table 13.2.3.1). The result could be termed a risk matrix, but the hazard for each cell occurs with 100% probability.

Table 13.2.3.1 displays DS2 for all v at f_{diff} = 0.0 m. The impact of high v when f_{diff} effectively equals zero is addressed in Section 13.2.5.

Table 13.2.3.1: Example of a Risk Matrix

(Using the DS-to-LEP conversions from Table 13.2.1.1)

		f_{diff} (m)						
		no flood water contact	0.0 (0.0-0.25)	0.5 (0.25-0.75)	1.0 (0.75-1.25)	1.5 (1.25-1.75)	2.0 (1.75-2.25)	2.5+ (2.25+)
v (m/s)	0.0	0%	50%	50%	78%	87%	99%	100%
	0.5	0%	50%	50%	81%	95%	99%	100%
	1.0	0%	50%	50%	81%	95%	99%	100%
	1.5	0%	50%	58%	81%	95%	100%	100%
	2.0	0%	50%	58%	84%	95%	100%	100%
	2.5	0%	50%	58%	84%	99%	100%	100%
	3.0	0%	50%	58%	87%	99%	100%	100%
	3.5	0%	50%	78%	87%	99%	100%	100%
	4.0	0%	50%	78%	95%	100%	100%	100%
	4.5	0%	50%	81%	95%	100%	100%	100%
	5.0	0%	50%	81%	99%	100%	100%	100%
	5.5	0%	50%	84%	99%	100%	100%	100%
	6.0	0%	50%	84%	100%	100%	100%	100%
	6.5	0%	50%	87%	100%	100%	100%	100%
	7.0	0%	50%	87%	100%	100%	100%	100%
	7.5	0%	50%	95%	100%	100%	100%	100%
	8.0	0%	50%	95%	100%	100%	100%	100%
	8.5	0%	50%	95%	100%	100%	100%	100%
	9.0	0%	50%	99%	100%	100%	100%	100%
	9.5	0%	50%	99%	100%	100%	100%	100%
	10.0	0%	50%	99%	100%	100%	100%	100%
	10.5	0%	50%	99%	100%	100%	100%	100%
	11.0+	0%	50%	100%	100%	100%	100%	100%

Risk matrices in the form of Table 13.2.3.1 provide indicative results of floods which would produce large losses. The practical value is limited because floods would not be expected to give a uniform f_{diff}-v combination over a case study site. $h(f_{diff},v)$ from Tables 13.2.2.1 to 13.2.2.5 is required:

$$\sum_{\text{All } f_{diff},v} \text{LEP}(f_{diff},v) \times h(f_{diff},v) = \text{LEP index as a measure of risk}$$

This LEP index represents the percent of the value of all residences in Canvey Island which is lost due to the flood scenario.

13.2.4 Results for Canvey Island Flood Loss Scenarios

Forty loss scenarios were explored through combining the five sea wall breach scenarios (Section 13.2.2) with two flood rise speeds and four residence variations. The terminology used for these scenarios is:

Normal Internal Flood Resistance: Typical residences which have not been altered from their current state (e.g. Table 13.2.1.1).

High Internal Flood Resistance: Measures have been taken to increase the residence's flood resistance (e.g. Table 13.2.1.2).

Castle View: The breach is adjacent to Castle View (Figure 13.2.2.1). A 120 m to 220 m wide golf course lies between the sea wall and the first line of residences.

Newlands: The breach is adjacent to Newlands (Figure 13.2.2.1). The residences lie immediately behind the sea wall, 10-20 m away.

Newlands Buffer Zone: The breach is adjacent to Newlands. In the model, residences within 100 m of the breached sea wall are replaced with thick vegetation, thereby creating a flood buffer zone.

Normal Residence Walls: The residence walls have $W \leq 4.0$ m or $\lambda > 0$ (i.e. they are not wide or they support some of the residence's weight).

Weak Residence Walls: The residence walls have $W \geq 5.0$ m and $\lambda = 0.0$ (i.e. they are wide and they support little of the residence's weight).

Fast-rise Flood: Assumes that the maximum f given by Brown (2003) equals f_{diff}.

Slow-rise Flood: Assumes that $f_{diff} = 0.0$ m always.

This difference was achieved by assuming that DS2 would be the highest DS which could be experienced for a slow-rise flood. To avoid altering the vulnerability matrices from Chapter 12, the LEPs for DS3, DS4, and DS5 are set to DS2's LEP. Tables 13.2.4.1 and 13.2.4.2, based on the fast-rise flood Tables 13.2.1.1 and 13.2.1.2 respectively, illustrate the slow-rise flood scenarios. Discussion on the validity of this approach is deferred until Section 13.2.5.

Table 13.2.4.1: Slow-Rise Flood LEPs for a Typical Residence

DS	0	1	2	3	4	5
LEP	0%	2%	50%	50%	50%	50%

Table 13.2.4.2: Slow-Rise Flood LEPs for a Residence with High Internal Flood Resistance

DS	0	1	2	3	4	5
LEP	0%	2%	10%	10%	10%	10%

The results are listed in Tables 13.2.4.3 to 13.2.4.8.

Table 13.2.4.3: LEPs for Castle View and Normal Residence Walls

Scenario	Flood (Hazard)	Residences (Vulnerability)	LEP Index
50 m breach	fast-rise	normal internal flood resistance	8%
50 m breach	fast-rise	high internal flood resistance	5%
50 m breach	slow-rise	normal internal flood resistance	6%
50 m breach	slow-rise	high internal flood resistance	1%
200 m breach	fast-rise	normal internal flood resistance	39%
200 m breach	fast-rise	high internal flood resistance	25%
200 m breach	slow-rise	normal internal flood resistance	30%
200 m breach	slow-rise	high internal flood resistance	6%

Table 13.2.4.4: LEPs for Castle View and Weak Residence Walls

Scenario	Flood (Hazard)	Residences (Vulnerability)	LEP Index
50 m breach	fast-rise	normal internal flood resistance	8%
50 m breach	fast-rise	high internal flood resistance	5%
50 m breach	slow-rise	normal internal flood resistance	6%
50 m breach	slow-rise	high internal flood resistance	1%
200 m breach	fast-rise	normal internal flood resistance	39%
200 m breach	fast-rise	high internal flood resistance	26%
200 m breach	slow-rise	normal internal flood resistance	30%
200 m breach	slow-rise	high internal flood resistance	6%

Table 13.2.4.5: LEPs for Newlands and Normal Residence Walls

Scenario	Flood (Hazard)	Residences (Vulnerability)	LEP Index
50 m breach	fast-rise	normal internal flood resistance	12%
50 m breach	fast-rise	high internal flood resistance	5%
50 m breach	slow-rise	normal internal flood resistance	10%
50 m breach	slow-rise	high internal flood resistance	2%
200 m breach	fast-rise	normal internal flood resistance	47%
200 m breach	fast-rise	high internal flood resistance	28%
200 m breach	slow-rise	normal internal flood resistance	36%
200 m breach	slow-rise	high internal flood resistance	7%

Table 13.2.4.6: LEPs for Scenario 4, Newlands and Weak Residence Walls

Scenario	Flood (Hazard)	Residences (Vulnerability)	LEP Index
50 m breach	fast-rise	normal internal flood resistance	12%
50 m breach	fast-rise	high internal flood resistance	5%
50 m breach	slow-rise	normal internal flood resistance	10%
50 m breach	slow-rise	high internal flood resistance	2%
200 m breach	fast-rise	normal internal flood resistance	47%
200 m breach	fast-rise	high internal flood resistance	28%
200 m breach	slow-rise	normal internal flood resistance	36%
200 m breach	slow-rise	high internal flood resistance	7%

Table 13.2.4.7: LEPs for Newlands Buffer Zone and Normal Residence Walls

Scenario	Flood (Hazard)	Residences (Vulnerability)	LEP Index
200 m breach	fast-rise	normal internal flood resistance	24%
200 m breach	fast-rise	high internal flood resistance	11%
200 m breach	slow-rise	normal internal flood resistance	21%
200 m breach	slow-rise	high internal flood resistance	4%

Table 13.2.4.8: LEPs for Newlands Buffer Zone and Weak Residence Walls

Scenario	Flood (Hazard)	Residences (Vulnerability)	LEP
200 m breach	fast-rise	normal internal flood resistance	24%
200 m breach	fast-rise	high internal flood resistance	11%
200 m breach	slow-rise	normal internal flood resistance	21%
200 m breach	slow-rise	high internal flood resistance	4%

13.2.5 Discussion and Interpretation of Canvey Island Flood Loss Scenarios

The following trends were observed:

- For all scenarios, the wider breach resulted in LEPs approximately three to five times higher than the narrower breach.
- For all scenarios, weak wall panels resulted in no apparent difference to the LEPs compared with normal wall panels.
- For all scenarios, the trend from highest to lowest LEPs was:
  -fast-rise flood, normal internal flood resistance
  -slow-rise flood, normal internal flood resistance
  -fast-rise flood, high internal flood resistance
  -slow-rise flood, high internal flood resistance
- The location of the breach changed the LEPs by 10%-50%.
- For the scenarios considered, the buffer zone with thick vegetation approximately halved the LEPs.

The factor of different locations becomes apparent when examining the flooded residential area (Table 13.2.5.1). The Newlands breach flooded more residential area on Canvey Island than the Castle View breach; however, Brown (2003) notes that the Castle View breach flooded a much larger total area than the Newlands breach. If undeveloped parts of Canvey Island were developed, a Castle View breach could have far higher LEPs. As well, the vegetated buffer zone reduced both the inundation area and the LEPs by 40%-50%.

Table 13.2.5.1: Percent Residential Area of Residential Canvey Island Flooded

	Castle View	Newlands	Newlands Buffer Zone
50 m breach	12%	21%	
200 m breach	59%	73%	41%

Weak rather than normal wall panels negligibly affected the LEPs. The reason is that the vulnerability matrices displayed few cells where the presence of weak panels influenced the damage outcome. The vulnerability matrices were presented at a resolution of 0.5 m for f_{diff} and 0.5 m/s for v. The transition for most residence walls from not failing to failing occurred over one 0.5 m f_{diff} interval; i.e., relatively rapidly at the resolution considered. This interval size is not sufficiently small to differentiate weak and normal wall panels.

The slow-rise flood LEPs are credible because the assumptions are generally realistic with one exception. The exception is that with finite f but $f_{diff} = 0.0$ m, v is assumed to never cause damage higher than DS2. Theoretically, this assumption is unrealistic. Practically, few floods in flat areas such as Canvey Island produce high v without noticeable f_{diff}. In the rare cases where a flood occurs with high v and negligible f_{diff}, higher LEPs than estimated here would occur. Therefore, the slow-rise flood LEPs presented here must be taken as a lower bound.

The fast-rise flood LEPs must be considered as only an absolute upper bound of the losses to be expected for each scenario. The two assumptions necessary for the fast-rise flood scenario were:

- $f_{diff\,max}$ equals the f_{max} given by the models.
- $f_{diff\,max}$ coincides in time with v_{max} for every residence.

Therefore, the LEPs for the fast-rise flood scenarios represent only an upper bound rather than a realistic scenario. With the slow-rise flood LEPs providing a lower bound and the fast-rise

flood LEPs providing an upper bound, actual LEPs for a breach scenario would be expected to fall between these two values, but usually closer to the slow-rise flood LEPs.

13.2.6 Mitigation Options Based on Canvey Island Flood Loss Scenarios

As expected, increasing the internal flood resistance of residences assists in reducing losses. The challenges of encouraging individuals to take such action are noted in Section 13.4.4.

The buffer zone reduced the inundation area and, generally, maximum f_{diff} and maximum v experienced by residences. The use of a buffer zone with thick vegetation appears to substantially reduce losses.

The wider breach was four times as wide as the narrower breach. Losses from the wider breach were approximately three to five times higher than losses from the narrower breach. From two breach widths, losses appear to increase at the same proportion as the breach width increases. At the first-order, neither an advantage nor a disadvantage appears for several narrower breaches in a sea defence compared to one wider breach of equivalent width.

Fast-rise floods have the potential for causing many more losses than slow-rise floods. A secondary line of smaller defences behind the main sea wall or a buffer zone with a slow upwards gradient away from the main sea wall could retard the advance of the flood wave enough to ensure that a slow-rise flood occurs. Such solutions are not immediately recommended here, because other factors must be considered. For example, introducing an upward slope in a buffer zone would entail some form of downward slope or sudden drop before the residences. Similarly, a secondary line of defence which also breaches may not change the ultimate flood outcome.

A heavily vegetated buffer zone could be an appropriate approach for preventing a fast-rise flood. Vegetation, though, may become debris hazards in floods and wind storms. As well, the long-term viability of a heavily vegetated buffer zone is questionable because eventually residents or developers may wish to develop more residences or commercial premises, may complain about its unsightly appearance, or may wish to turn it into a recreational area, such a playground or golf course.

The assumption of residence homogeneity across Canvey Island was made to illustrate the LEP method. This assumption may be appropriate for some locations, including Canvey Island, but would not apply to all flood-vulnerable locations. One way of overcoming this assumption would be to divide the case study site into smaller communities in which the residence distribution is shown to be homogeneous. Flood models must then provide $h(f_{diff},v)$ for each of these smaller communities and the LEP method may be applied for each smaller community. Summing over all smaller communities, with each community's LEP index weighted by its area relative to the area of the entire case study site, would yield the LEP index for the entire case study site. Similar area-weighted summations could be used over several case study sites to produce a LEP index for a region, such as coastal, eastern England between the Humber and Thames Estuaries.

Many other assumptions were necessary to reach the results, so caution is warranted in interpreting the options presented in this Section. Policies promoted and decisions made should always take into account the uncertainties and unknowns. In particular, these analyses were for modern residences, mainly post-1960, because those residences dominate Canvey Island and Hull. Nevertheless, a method has been developed and illustrated to integrate the output from hydrodynamic flood models with residence vulnerability profiles in order to produce a focused output of quantitative risk. This quantitative risk analysis would provide an important input into flood management recommendations related to structural and community design, such as those explored in Section 13.4.

13.2.7 Conclusions

The quantification of risk through LEPs yields insights which assist in understanding and analysing flood risk to residences. The approach suggested in this Section is one analysis amongst many possibilities. Specific definitions of quantitative hazard and quantitative vulnerability were suggested and applied, but other forms may be feasible and could be adapted for specific circumstances encountered, such as residences different to those observed in the case study sites.

The calculations here illustrate the usefulness of the vulnerability matrix tool developed in this dissertation, suggest how the results from this dissertation may be integrated with others' work, and quantify risk in a manner consistent with accepted definitions and from which

lessons may be extracted. This quantitative definition of risk is further explored in Section 13.3.

13.3 Risk Quantification: Definitions

13.3.1 Risk Definition

From Section 2.5, the risk equation is:

$$\text{risk} = \sum_{\text{All residences}} \int_{\text{All values of the hazard parameter}} [\text{(hazard parameter value exceedance probability)} \times \text{(vulnerability to that hazard parameter value)}] \text{ integrated with respect to the hazard parameter.}$$

The LEP method of calculating quantitative risk in Section 13.2 uses part of this definition. Summing over all possible hazard probabilities was not feasible because these data were not available from Brown's (2003) models. Summing over all residences was achieved yielding the following interpretation of risk as a LEP index:

If a specific flood event does occur,

$$\text{risk as a LEP index} = \sum_{\text{All } f_{diff}\text{-v combinations and residences}} \text{(area over which } f_{diff}\text{-v hazard is experienced)} \times \text{(vulnerability of that area expressed as a LEP)}$$

The core elements of risk as mathematical expectation, as discussed in Section 2.2.1 are still present. The f_{diff}-v combinations are defined discretely, 0.5 m intervals for f_{diff} and 0.5 m/s intervals for v, permitting the summation. This risk definition is useful for comparing scenarios, as seen in Section 13.2. The risk environment, though, is not completely described because an expression of the timeframe and the probabilities of specific scenarios occurring are not provided.

Despite the gaps which exist in fully defining the hazard, the objectives of this dissertation (Section 1.3) were focused on better descriptions and understanding of risk through exploring physical vulnerability. The aspects of vulnerability selected have been fully explored and used to evaluate risk, hence this scenario approach is appropriate in the context of this dissertation. As shown with the LEP method, the consequences of choosing mitigation

options may be examined through scenarios. Such scenarios could also include the impact of choosing different definitions of vulnerability.

13.3.2 Vulnerability Definitions

Section 13.2 illustrated the LEP method with one choice of vulnerability definition: the percent value of structure, contents, finishes, and systems lost in a flood event. Other suggestions for definitions were provided. Some of these definitions determined value through so-called "objective" methods, predominantly either the market value or the amount of money which would be paid. Several challenges exist with keeping this approach accurate and robust.

The replacement value of damaged residences is different from the repair value. Whether an occupant, or perhaps their insurer, chooses replacement over repair would affect the ultimate loss. This choice may not be consistent within a community. Different methods of repair and purchase of different replacements would also affect the consistency of the DS-to-LEP conversion. Tables 13.2.1.1, 13.2.1.2, 13.2.4.1, and 13.2.4.2 could be assumed to provide a mean value for DS-to-LEP conversions.

Similarly, the number of storeys (j) in the residence may affect the DS-to-LEP conversion. For example, if the slow-rise flood depth (f) is 2.5 m, LEP for DS2 would be expected to be much higher for j = 1 than for j = 3. The reason is that 2.5 m of flood water would inundate most of the one-storey residence, perhaps making replacement preferable to repair, but would inundate only the ground floor of the three-storey residence. For fast-rise floods, the vulnerability matrices for j = 1 do not display DS2 at $f_{diff} > 0.5$ m. At $f_{diff} = 0.5$ m, the difference between j = 1 and j = 3 for typical residences' LEPs is less than for f = 2.5 m. The reason is that most of each residence is left intact, yet boilers and electrical systems, amongst other high-cost items, would likely be flooded irrespective of j. For residences with high internal flood resistance, j might have far greater impact on the DS-to-LEP conversion. High internal flood resistance, however, could preclude j = 1 or could require the residence to be raised 1 m from ground level, thereby making the DS-to-LEP conversions more even.

Despite the ambiguities in converting DS2 to a LEP, DS5 would be close to 100% for all j, irrespective of the flood rise speed. The discrepancies at DS2, and potentially other DS levels, could be overcome by suggesting again that the DS-to-LEP conversions could be

considered approximate mean values. To achieve better risk quantification, an extension to this study could be to produce vulnerability matrices for each A-j type which have LEPs rather than DS values for each f_{diff}-v combination. This extension would reduce the applicability of the vulnerability matrices because different DS-to-LEP conversions—for example, for different levels of internal flood resistance—could not be tested without revising all the vulnerability matrices.

The sensitivity of the results to the DS-to-LEP conversions may be estimated. Canvey Island residences are 25% 1-storey, 69% 2-storey, and 6% 3-storey (Chapter 6). For a flood with f_{diff} = 0.5 m and v = 0.5 m/s inundating 50% of the residential area of Canvey Island, results of varying the DS-to-LEP conversions with j are listed in Table 13.3.2.1. The increase in risk from j = 1 being damaged more than j = 2 is slightly offset by the decrease in risk from j = 3 being damaged less than j = 2. Until extreme variance with j is reached, the risk as a LEP index for Canvey Island does not change markedly. Thus, the approximation in the vulnerability definition of j-independent DS-to-LEP conversions is a minor source of uncertainty. Further investigation of this aspect could be a task for refining the LEP method.

Table 13.3.2.1: Sensitivity of DS-to-LEP Conversions Varying With j

Scenario	j	DS-to-LEP Conversion	LEP for Canvey Island
No variance with j.	1	50%	25%
	2	50%	
	3	50%	
Small variance with j.	1	60%	26%
	2	50%	
	3	40%	
Large variance with j.	1	80%	28%
	2	50%	
	3	30%	
Extreme variance with j.	1	100%	30%
	2	50%	
	3	15%	

Alternatively, the DS-to-LEP conversions could be defined as probability distributions, such as a normal distribution. A specific flood scenario could be simulated multiple times in order to develop a probability distribution for the calculated risk as a LEP index.

The implicit assumption for these calculations is that given a specific flood event or f_{diff}-v combination at a particular residence, the DS level and hence the LEP or LEP probability

distribution would always be the same. Occupants have the ability to influence their personal LEP through pre-flood actions such as:

- Moving furniture and appliances upstairs to reduce losses.
- Sealing entry points to the residence because of their belief about the impact that sealing would have on losses.
- Moving old furniture and appliances downstairs to take advantage of their insurance company's "new for old" policy.
- Opening entry points into the residence because of a belief that losses would be minimised or to take advantage of their insurance company's policy of repainting and refinishing a damaged residence.

Some of these actions transfer the loss, potentially benefiting the occupant, rather than necessarily altering the LEP. The effectiveness of such actions usually depends on the timing, nature, and accuracy of warnings or flood information received. The appropriateness of these actions also varies, since moving contents into harm's way could be fraud and might invalidate an insurance policy.

The distribution of costs and benefits amongst different sectors of society also influences the meaning of vulnerability. For example, the loss to the global insurance or reinsurance industry (or both combined) of a flood event in Hull would be different from the loss to all of society, to the economy, to the U.K., to Humberside, or to individuals. Many flood events are beneficial to specific commercial sectors, at least over a certain time scale, through the brisk trade in repair works coupled with possible price inflation locally on materials and services needed. Meanwhile, individuals might lose uninsured items, invaluable and irreplaceable items such as photographs, or items which have different economic values to different individuals.

One definition of vulnerability in Section 13.2 suggested using "value to owner" as the basis for measurement. Data for subjective valuations are difficult to collect and would rarely be robust through time. The principal advantage is that the loss is valued by the individuals most directly affected. One drawback, as with most other methods, is that the effects on sectors other than the valuer would not be considered in the analysis.

Defining the method for quantifying vulnerability is challenging. Since no analysis could be completely comprehensive, different definitions may be appropriate for different

circumstances. Using different definitions would be useful for checking the sensitivity of results as well as for analysing options from different viewpoints.

13.3.3 Conclusions

Risk and vulnerability can be quantitatively defined and analysed using the research from this dissertation. Scenarios and different definitions may be compared. Considering different definitions of vulnerability indicates that aspects of vulnerability are subjective and qualitative, which in turn impacts the meaning of the risk calculation. As noted in Section 2.2.3, Helm (1996), Lewis (1999), and Sayers *et al.* (2002) suggest cautions in interpreting quantitative risk because the value presented does not describe the entire risk in reality. Analysis strategies and decision making tools must factor in these non-quantitative aspects. Some examples of these methods are discussed in Section 13.4.

13.4 Analysis Strategies and Decision Making

13.4.1 Introduction

The research in this dissertation can provide qualitative as well as quantitative input into analysis strategies and decision-making processes regarding the management of flood damage to modern residences in coastal, eastern England. This Section highlights some of the key issues and illustrates factors to consider in applying the research in this manner. The focus is on strategies to use for analysis and the options available rather recommending specific options for all circumstances. Lessons which may be learned from this dissertation are articulated and provide guidance for an individual's, community's, or sector's choice for managing flood damage to residences.

Despite extensive literature on residence flood vulnerability management from countries such as the U.S.A. (e.g. USACE, 1993) and Australia (e.g. BTRE, 2002), U.K. sources are used in this Chapter to ensure that the advice is as practical as possible for this dissertation's study area. As with most work on U.K. residence flood damage (see Chapter 2), the flood action of most concern to sources on residence flood vulnerability management is water contact due to slow-rise flood depth (f). To maintain the focus of this dissertation established in Chapter 3, this Section interprets this literature in the context of the flood actions of the pressure differential (ΔP) from depth differential (f_{diff}) and velocity (v).

13.4.2 Issues of Concern

Because residence vulnerability management choices are rarely straightforward, the appropriate starting point is some issues of concern. This Section places certain aspects of this dissertation's research into a wider context and notes where caution may be warranted in applying the results.

Analysing wall failure (Chapter 9) and glass failure (Chapter 10) both required a choice of a failure mechanism. The choice for wall failure was of a binary system. On one side of a f_{diff}-v curve, the wall fails while on the other side, it does not fail. In contrast, glass failure was analysed probabilistically. The f_{diff}-v charts displayed curves for which the probability of failure is the same for any f_{diff}-v combination on the curve.

These two methods were selected from literature searches which indicated the most accepted approach to the respective problems. A discrepancy occurs in that the two residence components were examined with different analysis methods. Alternate assumptions were feasible. For example, masonry properties such as density (ρ_b) or thickness (þ) could have been assigned probability distribution functions. Wall failure would be probabilistic, defined by the probability that the set of (probabilistic) masonry properties yields a wall which fails under a given f_{diff}-v combination. Similarly, glass failure could be selected as deterministic by choosing a specific tensile strength (Ω) rather than a range to which probabilities were assigned.

These alternatives are not supported by the literature. Adequate data do not exist to select the probabilistic density functions for masonry properties or a specific Ω for glass.

This study combines the fields of structural engineering, for wall failure, and material science, for glass failure, in the practical application of residence failure under specific flood actions. Consistency of method is not necessarily an appropriate objective, but awareness of and explanation for the differences assist in tackling the fundamental, physical explanation of what happens to a residence under different f_{diff}-v combinations. Uncertainties in the analysis and differences between estimations and observed results may be better understood. In this case, the different methods emerge from the literature.

The analysis presented in this dissertation, including the vulnerability matrices (Chapter 12), is thus credible only if the assumption is made that the literature has provided appropriate guidance. If selection of either the analysis method or the failure mechanisms were inappropriate, then the vulnerability matrices and subsequent application may produce different results.

Either full-scale tests or further observations of f_{diff}-v flood damage to residences, along with measurements of the f_{diff}-v combinations which occurred, would be needed to determine the accuracy and precision of the results presented in this dissertation. Obtaining such data presents many challenges, but would be the principal method for further refining and justifying the approach taken in this dissertation.

Another issue which might be best resolved through empirical or laboratory studies is interaction amongst failure mechanisms. For example, as one leaf of a cavity wall starts to fail, doors and windows may be subjected to extra stress. One may fail when glass or another component fails. If the wall fails slowly enough, the increase in flood rise rate through the failed door or window could be enough to reduce the pressure differential on the wall, preventing collapse. Such potential interactions and sequences of events are speculative, but would be challenging to fully analyse without experimentation or observation.

Other flood actions in case study sites require further exploration because, as noted in Chapter 3, they may have significant impact when they manifest. An example is wind-generated waves. Based on Lloyd and Harper (1984; see also Chapter 3), any wave of height over approximately 0.4 m breaking directly on a property has a high probability of shattering windows (DS3) and would often be expected to cause walls to collapse (DS4). With adequate wind speed, such waves would be likely over flooded areas of Canvey Island and could possibly affect up to 25% of the residences examined in Hull (see also the discussion in Section 4.3). Waves thus have the potential to cause significant damage, even following a slow-rise, low-velocity flood. Debris could have significant effects on residence damage, particularly when the debris would include, for example, propane tanks or vehicles with full petrol tanks, potentially causing explosions.

Due to the large local variation of these phenomena, the uncertainties are so large that predicting their effects is not feasible. Being aware of them, and the uncertainties which they entail, is important.

Finally, the results in this dissertation have limited applicability due to the focus on modern residences in England. Timber-frames in Suffolk and knarled flint in Norfolk, for example, are common in flood-vulnerable areas of coastal, eastern England. Structures made from reinforced or non-bonded masonry are also not covered by this dissertation. Although the specific results and vulnerability matrices in this dissertation could not be readily transferred elsewhere, the techniques, methods, and lessons learned could apply to many other locations and residence types.

13.4.3 Options for Residence Flood Vulnerability Management

Numerous authors provide material on options for residence flood vulnerability management, before, during, and after flood actions impact a residence (e.g. Bramley and Bowker, 2002; BRE, 1997; BRESL, 1996; CIRIA, 2001-2; Dixon, 2001; DTLR, 2002; EA, 2001-2; EA/CIRIA, 2001a; EA/CIRIA, 2001b; floodforum.net, 2002; Lewis, 2001-2; NFF, 2002; SEPA, 2001-2). Advice commonly focuses on:

- Dry flood proofing a residence, i.e. trying to keep flood water out such as by sealing openings or raising the residence.
- Wet flood proofing a residence, i.e. permitting flood water to enter but minimising damage such as by using flood-resistant materials and finishes.
- Relocating a residence or community.
- Designing a community to prevent flood actions impacting residences, such as drainage patterns which discourage surface water ponding near residences and street layout which prevents large water velocities impacting residences.
- Otherwise altering the flood hazard parameters which affect residences and communities through:
  -hard flood defences such as dams, levees, and walls; and
  -soft flood defences such as wetlands and parks used to store excess water.
- Efficient and effective return of the residence to a pre-flood, or better, state after the flood event.

These sources offer comprehensive advice on the possibilities which exist but rarely develop tools for determining which option may be the most appropriate in given circumstances. As with many decision-making dilemmas, defining the most appropriate option depends on the criteria being considered and the most important criteria according to the judge. The next

three Sections illustrate potential analysis strategies which could be used, irrespective of the perspective taken.

13.4.4 Analysis Strategies for Individual Residences: Sealing

The analysis of Flood Rise Rate provides further insight than indicated in Chapter 8, particularly when considered in the context of wall failure (Chapter 9) and glass failure (Chapter 10). The results from these three inputs yield not only the vulnerability matrices (Chapter 12) but also a potential decision-making tool.

During a flood, an occupant's inclination is often to seal a residence in order to prevent flood water infiltration and DS2. The goal of sealing is questionable because Chapters 9 and 10 indicate that glass and wall failure may occur at low $f_{diff} \approx 1.0$ m. Since glass failing leads to DS3 and wall failure leads to DS4 or DS5, sealing to prevent DS2 may result in worse damage than without sealing. A residence may need two options: to seal if ΔP_{max} is forecast to be below a certain level and to unseal if ΔP_{max} is forecast to be above a certain level. If the forecast is uncertain or falls within the range where the choice to seal or not to seal is ambiguous, then this approach presents challenges.

The simplified decision-making dilemma in such a circumstance is illustrated in Table 13.4.4.1. Sealing yields outcomes of low loss (DS1), moderate loss (DS2), or high loss (DS3, DS4 or DS5) whereas not sealing always yields the outcome of moderate loss (DS2). Two other issues are involved to reach a recommendation: the ease of implementing a decision and the ease of changing the outcomes in Table 13.4.4.1.

Table 13.4.4.1: Decision-making Matrix for Sealing if ΔP Forecast is Uncertain

Flood ↓ Decision →	Prevent Infiltration (Seal)	Permit Infiltration (Do Not Seal)
f_{diff}-v combination would break glass or walls.	Outcome is DS3, DS4 or DS5.	Outcome is DS2.
f_{diff}-v combination would not break glass or walls.	Outcome is DS1 or DS2.	Outcome is DS2.

Regarding the ease of implementing a decision, permitting infiltration is much easier than preventing infiltration. As noted in Chapter 8, typical residences have a non-negligible flood infiltration rate and flood rise rate. In fact, residences are designed with enough leakiness for ventilation. Completely eliminating infiltration requires surrounding a residence with a

continuous, impermeable barrier, such as plastic sheeting (e.g. see EA/CIRIA, 2001b). Alternatively, preventing infiltration through only the main routes such as doors and air bricks may be attempted by using sandbags. Home-made or commercially-available flood barriers which cover openings and seal around the edges are generally more effective than sandbags (EA/CIRIA, 2001b). Examples of companies in England which sell flood barriers for residences are Antiflood.com, Floodbar UK Ltd., Floodguards International Limited, and Floodskirt Ltd. These products require investment, knowledge on proper use, maintenance, and adequate warning. Whilst in operation, inconveniences occur, such as difficulty in entering or leaving the residence and lack of ventilation to the wall's cavity.

Permanently reducing sources of infiltration is a longer-term solution. Postal flaps could be raised or completely removed and replaced with post boxes. Residences could be built with concrete rather than timber floors, on the assumption that rising groundwater would not be a threat to breaking the concrete floor; otherwise, potential DS4 from a broken floor would need to be weighed against DS2 from permitting infiltration. All doors and windows could be weather-stripped which has further advantages for energy efficiency and weather-tightness. These solutions, though, slow down the flood rise rate. A flood lasting more than a few hours, a usual situation, would still inundate the inside of the residence causing DS2. Entirely eliminating infiltration is therefore not a feasible solution for altering the outcomes in Table 13.4.4.1.

In contrast, permitting infiltration is simple: do nothing. The process could be facilitated by lowering postal flaps and opening doors or other openings. Opening doors and windows has security disadvantages yielding the possibility that occupants may be encouraged to remain in their residence leading to an increased risk to life.

Regarding the ease of changing the outcomes in Table 13.4.4.1, vulnerable glass may be boarded up to prevent DS3, but occupants generally have no simple solution for preventing DS4 or DS5. Substantial investment and expert advice would be required. In contrast, reducing damage due to infiltration is relatively straightforward. Immediate actions include storing valuables on upper storeys and removing carpets from ground floors. If an appropriate plan has been made, including planning to have help if needed, and if a flood warning is received in time, most contents could be removed from danger. Living in bungalows or ground floor flats makes this process more challenging.

Longer-term solutions (CIRIA, 2001-2; Dixon, 2001; DTLR, 2002; EA/CIRIA, 2001a; EA/CIRIA, 2001b; floodforum.net, 2002; Kelman 2001; Lewis, 2001-2) for ground floors include raising electric and telephone cabling so that flood water must rise higher to reach and damage them; installing drains with one way valves in ground floor rooms; and using swimming pool tanking (plaster) rather than normal paint. Residences could be raised farther above ground level (i.e. increasing b of components), preferably during construction but feasibly for retrofitting.

Flood management, however, should not be considered in isolation because solutions may have drawbacks. Some swimming pool tanking results in off-gassing, posing a health risk (floodforum.net, 2002). Improper installation or maintenance of drains and valves could lead to sewage backing up into the residence. Raising the entire residence could make life difficult for elderly or disabled occupants. Then, a valid question is the reason for vulnerable individuals settling in flood-vulnerable locations. Conversely, some solutions have dual advantages. Raising electricity sockets may assist in preventing babies or toddlers from sticking metal objects into outlets.

Irrespective of the challenges involved, multitudinous solutions and options exist for reducing damage due to flood water infiltration. Given the low f_{diff}-v combinations required for DS3, DS4, and DS5, permitting water to enter a residence may be simpler, longer-term, safer, and more effective than attempting to decide when to seal and ensuring that sealing is completed properly.

Whether or not occupants would accept flood water entering their residence, and would adapt their lifestyle and properties to make this option work, is a difficult sociological question. Nonetheless, it is frequently suggested (e.g. BRESL, 1996; EA/CIRIA, 2001b; floodforum.net, 2002; Kelman, 2001; Lewis, 2001-2) and has been implemented for several residences in England (e.g. Dixon, 2001; DTLR, 2002). This technique is not new. Lewis (1979) notes that traditional dwellings in Chiswell in southern England were built assuming that water would enter during floods: "The older cottages had stone or well-seasoned timber floors with flood ducts beneath. Occasional sea flooding was an accepted fact of life." Furthermore, as described in Chapter 9, DTLR (2002) and EA/CIRIA (2001b) advise limits on the height to which properties should be sealed against flood water entry because otherwise "keeping water out of your property can do more harm than good" (EA/CIRIA,

2001b). These references indicate the viability of permitting, and perhaps encouraging, flood water entry plus the need to make this option more socially acceptable.

The importance of this issue arises because the vulnerability matrices (Chapter 12) were dominated by flood rise rate yielding DS2 and wall failure yielding DS4 or DS5. This result is partly an artefact of the problem defined and assumptions made. For example, considering only ΔP before infiltration has occurred suggested that window failure need not be examined in detail due to windows' heights above ground. The implication is that when considering vulnerability mitigation options, the priorities rest with infiltration and wall failure, as glass failure does not appear prominently in the vulnerability matrices.

Table 13.4.4.1 shows that the DS4 or DS5 outcome is partly linked to the choice made regarding infiltration. The flood failure flowchart (Figure 11.2.3.1) therefore displays the sequence in which mitigation options should be considered because decisions in one box would influence decisions in subsequent boxes. The flood failure flowchart combined with the vulnerability matrices provides a useful tool for determining the order in which mitigation options should be chosen.

This discussion suggests that the first decision should be on infiltration: to seal or not to seal. Then, a decision would be necessary on whether or not to strengthen walls, such as by increasing the density or thickness of the masonry units. Different options could be selected and the implications compared. Strengthening masonry walls would be expected to be economically cheaper and easier in constructing new residences than in retrofitting existing residences, a fact which would emerge through such an analysis. Choices such as eliminating or adding large, low glass panes would be the next factor to consider. Similarly, decisions' impacts may be tested by assuming that a mitigation option is enacted, altering the vulnerability matrices accordingly, and running loss simulations with a technique such as calculating LEPs (Section 13.2).

Throughout such a process, many assumptions are necessary, so interpretation of results should always consider the uncertainties and unknowns. In particular, the trend of assumptions made throughout this dissertation has been generally to choose a worst-case scenario. This approach is somewhat conservative because it indicates maximum possible damage and losses from given scenarios. Nevertheless, each assumption made has been

realistic and sensitivities were checked and described. Therefore, the scenarios presented in this dissertation tend to be at the worst-case end of the realistic range of possible events.

Reducing uncertainties and establishing exactly where this dissertation's scenarios rest on the range of realistic events require more field data. While extreme flood disasters are undesirable, opportunities for field work should be grasped to try to match the estimations with the reality.

13.4.5 Analysis Strategies for Individual Residences: Other

Residence vulnerability management decisions other than those related to sealing may be analysed in a fashion similar to the discussion in Section 13.4.4, although interrelationships between this Section and Section 13.4.4 are inescapable. As noted in Chapter 2, duration is often considered a flood hazard parameter. Duration may also be a flood vulnerability parameter in referring to the time required to turn a residence into a pre-flood, or better, state.

This post-impact duration is often many months for U.K. residences:

- DTLR (2002) provides examples of properties which required several months to dry out following flooding and, regarding groundwater flooding, notes that "Properties can still be underwater many months after the heavy rains that caused the flooding have passed".
- EA (2001a) suggests that for flooded residences "current time to re-occupy is typically 1 year".
- On floodforum.net (2002), the U.K. residences of Anne Bury, Colin Walsh, and "Mr. Ryder" were flooded in October 2001 and no one was expecting to move back in until at least six months after the event. Jemma Mclay-fortune [sic] of Ironbridge, England was flooded in September 2000 and re-occupied at the end of July 2001.
- CIRIA (2001-2) comments "A brick dries out at about [0.0254 m] a month so be aware that it may take several months for the brickwork to completely dry out" and "It is important to remember that if a building is seriously flooded it may be a case of several months before it becomes habitable again and it can be many weeks before it dries out".
- EA/CIRIA (2001a) notes that "It is difficult to estimate how long a property will take to dry out…In the case of acute flooding, be prepared for the process to take months rather than weeks".

This time is required for proper drying, cleaning, repair, and redecorating of all storeys affected. Improper rehabilitation of a residence following a flood may result in long-term damage or deleterious health effects due to mould (CIRIA, 2001-2; floodforum.net, 2002; NFF, 2002).

Some writers (e.g. Bramley and Bowker, 2002; CIRIA, 2001-2) on residence flood vulnerability mitigation imply that residences should not use materials that require replacement following a flood such as chipboard for floors or cupboard fittings. If this suggestion is accepted, then the material used must be made resilient and resistant to flood damage from most flood actions. Otherwise, if materials which require a months-long drying out period are used, then the cheapest, quickest, and easiest option may be to choose replaceable materials. Materials used in residences could be selected for efficient replacement just as they may be selected to be resilient and resistant to flood damage from most flood actions.

One concern is that of the waste produced. A solution which advocates efficient replacement over efficient drying and cleaning would produce large amounts of waste which might be contaminated by impure flood water.

One system which uses damageable materials without requiring replacement following each flood is to make the damageable components removable. Examples include detachable cupboard doors and loose-fitting rugs rather than fitted carpets (Bramley and Bowker, 2002; CIRIA, 2001-2; DTLR, 2002). The assumption is that the occupant will have sufficient warning time and ability to place all removable components in a flood-secure locale. If this locale is within the affected residence—an expected situation—and if DS5 occurs, then this action would be unhelpful because the entire residence and its contents would be lost (Table 13.4.5.1).

Table 13.4.5.1: Decision-making Matrix for Post-Flood Restoration of Residence Components

Option → / Flood damage ↓	Replace Components	Remove Components	Dry and Clean Components
DS2 or DS3	Outcome is replacement.	Outcome is removing and returning.	Outcome is drying and cleaning.
DS4	Outcome is replacement.	Outcome is removing and returning; possibly some replacement.	Outcome is drying and cleaning; possibly some replacement.
DS5	Outcome is replacement.	Outcome is replacement.	Outcome is replacement.

Some additional notes regarding the restoration option of "dry and clean components":

- Unanticipated contaminants could yield restoration problems.
- The quicker the process is commenced, the more effective this option becomes and the less time this option requires.
- If restoration is completed improperly, then long-term damage or major health problems may result (CIRIA, 2001-2; floodforum.net, 2002; NFF, 2002).
- If DS5 occurs, then the resilience of residence components to water contact damage may be irrelevant (Table 13.4.5.1) because they would need to be replaced.
- Replacement of a residence and its components always occurs at some time scale, irrespective of any flooding.

Some additional notes regarding the restoration solution of replacement:

- The extreme case of making the entire residence replaceable after every flood is unlikely to be viable due to factors such as cost and the occupants' psychological interest in maintaining their same home. Using cheap but easily replaced construction materials can lead to other problems including poor wind safety and inadequate thermal comfort.
- If DS5 occurs, then the items which were designated as replaceable would not be considered an extra loss (Table 13.4.5.1).

In Table 13.4.5.1, the outcomes cannot be changed (in contrast to Table 13.4.4.1), hence the decision may be influenced more by the ease of implementing an option. In particular, prominent factors for choosing the most appropriate option would be the initial cost of each option, the cost of each outcome, and the expected likelihood of each damage scenario. Criteria other than flood damage—including aesthetics, child safety, and component utility—

would be considered. Combinations are possible. DTLR (2002) does not state that flood-vulnerable materials should be avoided in kitchen storage units, but does suggest that they should be raised so that replacement is not necessary for shallow floods.

The cost and time for implementing an option could depend on the extent of local flooding and the choice made by nearby occupants. If all flooded occupants have chosen the same solution, then these goods and services will be in high demand, potentially raising the cost and lengthening the time to complete the restoration. On the other hand, a balance between the two solutions, and amongst different methods of implementing each solution, would place less demand on the needed goods and services. All residences may be restored more cheaply and more quickly than if only one solution were selected for all residences.

In restoring flooded residences, "resilient reinstatement" is promoted (ABI, 1999; floodforum.net, 2002). Resilient reinstatement involves increasing the flood resistance and resilience of a residence while repairing flood damage. All suggestions in this Section for altering a residence to better manage flood vulnerability could be part of resilient reinstatement. A suggestion for resilient reinstatement as a legal requirement is from BSD (2001) which considers improving a building "to a level no higher than that required by the Building Standards" following flood damage. Aspects of flood resistance and resiliency could be added to the Building Standards so that requiring resilient new developments automatically achieves resilient reinstatement for old developments when they are flooded (after Crichton, 2002).

The principle of resilient reinstatement could go beyond specific construction approaches. For example, a broad definition of resilient reinstatement could include rebuilding the residence in a less flood-vulnerable location. Alternatively, the occupants could abandon the site and purchase a property elsewhere. At the other extreme, resilient reinstatement could be that no changes to the residence are made. Instead, occupants accept that flood damage to their residence will occur frequently. By focusing on the needs of the occupant with respect to their residence, rather than the specific objective of preventing flood damage, resilient reinstatement becomes a social solution which often involves technical aspects, such as raising electricity sockets or the entire residence.

Another issue regarding the choice between drying/cleaning and replacement, which would also affect choices for resilient reinstatement, relates to the behaviour of masonry units in flooding. BRESL (1996) states that:

> Lightweight concrete with its relatively high moisture movement may expand on wetting and therefore shrink on drying. This may result in some cracking. Masonry materials in external walls can be prone to frost damage for about three weeks after flooding because moisture contents could be higher than under normal wetting and drying.

No indication is given regarding the seriousness of the damage. If cracking occurs to such an extent that significant remedial work is required, then Table 13.4.4.1 should consider this new factor. Allowing water to enter so that even internal walls become wet could result in significant repair work. The equivalent of DS4 may be reached by a slow-rise flood.

The focus on lightweight masonry materials for some of BRESL's (1996) comments suggests that using denser masonry units may be appropriate for residence flood vulnerability mitigation. Chapter 9 suggests increased masonry unit density as a possible solution to increase wall strength. In this instance, using denser material assists in two ways. In contrast, BRE (1997) notes that increasing masonry thickness increases drying time following a flood. In this instance, choosing the optimum thickness for masonry leaves yields a tradeoff between increased wall strength and reduced wall drying time.

One other issue regarding analysis strategies for individual residences is the question raised in Chapter 7 about whether or not doors and windows should be locked and, by extension, potentially shuttered, boarded up, or otherwise barricaded. If an occupant decides not to seal, as discussed earlier in Section 13.4.4, then ground floor doors and windows should be left as open as possible while balancing security concerns with the need for as high an infiltration rate as possible. If an occupant decides to seal, then doors and windows would be shut. Locking and barricading them would assist in preventing failure of these components. Locking and barricading, however, would further ensure that, if ΔP were large enough, walls would fail first yielding DS4 and DS5 rather than doors and windows failing yielding DS3.

13.4.6 Analysis of Strategies for Communities

This dissertation has illustrated the potentially significant damage to residences from f_{diff} and v. In developing community-wide flood management strategies, considering these hazard

parameters and how they manifest is important. Considering only f or assuming that the effects of f_{diff} and v are unimportant could underestimate losses. Analyses, such as cost-benefit analysis, may be skewed creating a misguided impression of appropriate solutions.

Although it is not a U.K. source, USACE (1996) provides a good example. USACE (1996) analyses the flood damage effects of a proposed levee by calculating the estimated difference between a depth-damage curve without the levee and a depth-damage curve with the levee. USACE's (1996) analysis assumes that as soon as the 6.68 m levee is overtopped, the damage would exactly equal the damage for a 6.68 m flood in the absence of the levee. This assumption is optimistic because:

- Without regular flooding at lower depths, the population would normally be less prepared for any flood which occurs, thus incurring more damage at higher flood levels (Etkin, 1999).
- The flood characteristics from a levee which overtops are expected to be different, likely with higher f_{diff} and higher v, than a flood of similar depth in the absence of any flood defence (Kelman, 2001).

Analysis of the impact of a structural flood defence on flood damage or losses in a community should take into account the hazard parameters f_{diff} and v, particularly regarding how the structural flood defence may exacerbate these parameters' effects.

The failure mode of a structural flood defence would partly determine the impact of f_{diff} and v. A breach or collapse of a structural flood defence would be expected to yield higher f_{diff} and v than overtopping. The time to these parameters' maxima would also vary amongst different defence failure scenarios.

Residence layout in a community also influences the extent to which f_{diff} and v manifest in a flood. Lewis (1999) notes that building layout, and even retaining damaged buildings, can assist in protecting communities from storm events. As one example in England, BB&V (2001) write:

> The extent, location and orientation of the various structures that have been constructed on the flood plain, both in Uckfield and in Lewes, made the effect of the 12th October 2000 flood worse than it would have been otherwise, by:
>
> - increasing the amount and rate of surface water run-off, thereby increasing flows;

- reducing the area available for flood storage, thereby increasing peak levels; [and]
- reducing the area available for flood flow conveyance, thereby increasing peak levels, contributing to rapid inundation and high flood velocities, and extending the period of flooding.

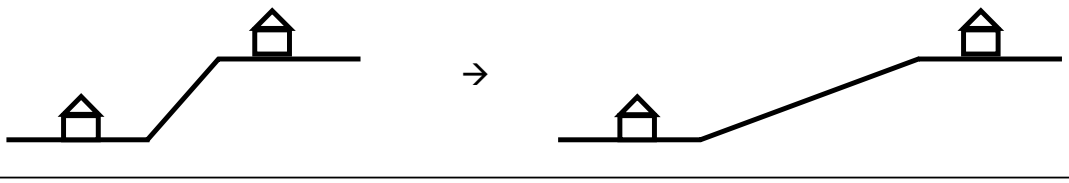
The work by Brown (2003) illustrates that hydrodynamic models have the capability of including residence shape, size, and orientation for analysing a flood's f_{diff} and v. This capability could be used in the design process, before the community is constructed, for exploring the community layout which would minimise ΔP from f_{diff}-v combinations. The hydrodynamic models, vulnerability matrices, and LEP approach could be implemented as a community design tool for minimising flood vulnerability.

Another example of structural flood defences potentially augmenting the damage from f_{diff} and v was witnessed in the Autumn 2000 floods in England (Table 1.1.1). Part of Keighley in Yorkshire was flooded by water from the River Aire. The structural flood defences of the adjacent but non-flooding River Worth held the water in the houses until the defences were deliberately breached, permitting the water to drain (Kelman, 2001). If this deliberate breaching had permitted the water to drain too swiftly, a residence could experience f_{diff} from the inside or high v imposed from the outside.

Awareness of the detrimental effects of f_{diff} and v should lead to flood management strategies which seek to diminish the impacts of these hazard parameters during a flood. No U.K. examples were found in the literature, but possible techniques for achieving this goal include:

- Smoothing slopes to prevent sudden transport of water from one section of a community to another section (Figure 13.4.6.1). Houses may need to built on the slope to avoid loss of development space. The upper numbers of St. Mary's Close in Hessle Foreshore, western Hull are built on a steep slope, indicating that this suggestion would not present a challenge for developers.
- Creating green spaces designed to become water storage areas during times of excess surface water. Underground storage areas could be considered. Drainage patterns and residence placement should lead surface water to storage areas without encountering

Figure 13.4.6.1: Smoothing a Slope to Potentially Diminish f_{diff} and v During a Flood



residences. This solution prevents water rising swiftly outside residences and does not place residences in the pathway of surface runoff.

More detailed investigation would be essential before implementing these suggestions and they may require contingency plans. For example, any green space used for water storage would have a finite capacity. An event may overwhelm that capacity. Finally, impacts outside the realm of flood management would need to be considered to ensure that diminishing the threat from f_{diff} and v would not exacerbate or create other concerns.

13.4.7 Conclusions

Section 13.4 has extended the examination of the most significant failure modes by considering methodological choices and decision-making processes which emerge from the analysis in this dissertation. This discussion has been mainly qualitative, thereby complementing the quantitative approach taken in Sections 13.2 and 13.3. Further insight has been gained into the innovative application of the this dissertation's research.

13.5 Conclusions

This Chapter has illustrated analysis techniques for using this dissertation's research. Possibilities for application to flood vulnerability management for modern residences in England have been described yielding decision-making tools and techniques. Both quantitative and qualitative approaches are discussed and the nature of risk is further explored. Due to conflicting objectives and uncertainties, the same recommendation is unlikely to emerge for every circumstance. A consistent basis for judgement and selection of options is now available and the potential consequences of a decision pathway may be described.

14. Conclusions

14.1 Achievements of This Dissertation

Section 1.3.1 defined this dissertation's overall objective, representing an original contribution to research, as demonstrating that significant improvements could and should be made in the manner in which society manages natural disaster risk. The work presented here has achieved this objective by:

- Illustrating the significance to residence flood damage of flood characteristics rarely examined in detail, namely flood depth differential (f_{diff}) and flood velocity (v).
- Illustrating the significance of structural failure in residences for flood damage and losses.
- Using the first two points to improve the profiling of physical vulnerability for residences at risk from flooding.
- Illustrating that the vulnerability profiles may be used for quantifying risk, for qualitative analysis, and for selecting vulnerability management options.

Achieving these four points necessarily involved the development of new knowledge and techniques for which calculations were completed at the first-order:

- Calculating flood rise rate (Chapter 8) applied established theory in a new manner.
- Analysis of wall failure (Chapter 9) used an established theory to analyse a load not previously considered.
- Analysis of glass failure (Chapter 10) involved developing an appropriate technique from the literature.
- These three analyses have been combined into the flood failure flowchart (Chapter 11) and applied to developing a new form of vulnerability profiling (Chapter 12).

Therefore, the work in achieving the main objective provides an original contribution to research.

The work to meet the main objective was focused through, and carried out in the context of, subobjectives:

- To demonstrate the main objective for the particular case of flood damage to residences in coastal, eastern England.
- To better describe, quantitatively and qualitatively, the risks for this case.

The first subobjective was met by choosing two case study sites, Canvey Island and Hull. A new method was developed for surveying and classifying residences. This method was

geared specifically towards assessing their physical vulnerability to floods. The data collected helped to meet the second subobjective to which the first-order calculations mentioned above also contributed. This approach demonstrated the usefulness of first-order calculations in estimating and quantifying residence failure modes during floods in order to achieve improved vulnerability profiling. Due to the observations in the case study sites, the results apply specifically to modern residences in England but the techniques and methods could be used for similar analyses for other locations and other residence types.

Finally, the tools which have been developed were implemented in an illustrative manner to demonstrate possible vulnerability management strategies and tools. A principal outcome was loss estimates for simulated storm surge scenarios on Canvey Island, presenting the loss equivalent percentage method of describing quantitative risk. Decision-making tools and analysis strategies were also discussed, indicating options available and potential justification for recommendations.

This dissertation started off by noting the heavy toll on society exacted by society's vulnerability to natural hazards. Flood disasters in coastal, eastern England were shown to be a prominent concern. This dissertation provides new knowledge and techniques which contribute to understanding, describing, and managing that vulnerability.

14.2 Recommendations for Further Study

14.2.1 Refining the Vulnerability Matrices and Flood Failure Flowchart

Despite the contributions of this dissertation, many knowledge gaps remain in this area and plenty of scope exists for reducing uncertainties in the calculations. One major task would be to refine the vulnerability matrices to a higher resolution, such as every 0.2 m for f_{diff} and every 0.2 m/s for v. Care must be taken in increasing the resolution because the relative uncertainty may increase due to the challenges of predicting the interactions of other flood actions, such as waves, or other natural hazards, such as wind.

More research is needed to incorporate the full range of flood actions into the vulnerability matrices. One approach to solving this problem would be to provide a list of scenarios, such as the presence of different contaminants, wind-generated waves, or scour. Each cell in the vulnerability matrices would have a statement mentioning the potential flood action and the

damage outcome if that action were to manifest. The vulnerability matrices use this approach in the cells which consider differences between the damage outcome if a door with large, low glass exists or if the residence has weak wall panels.

This dissertation covered the situation at the beginning of a flood when no water has yet entered the residence. Further development of the vulnerability matrices would also consider the pressure function induced by f_{diff} and v for which flood water has risen further up one side of a residence component than the other side (see Figure 3.2.2.1). Cases in which the flood level is between the top and the bottom of a glass panel were not considered in this dissertation because the empirical data are not available for the large-deflection stress induced by this pressure function. Scenarios in which water is trapped in a residence yielding a pressure differential from inside to outside could also be formally incorporated into the vulnerability matrices.

In addition to considering a wider set of variations for the flood actions and hazard parameters experienced by the residence, a wider set of interactions amongst residence components could be considered to refine the vulnerability matrices. This dissertation noted instances where interactions amongst residence components could exist, such as a bending mullion weakening window glass or distorting door frames increasing the flood rise rate. These interactions were not quantitatively analysed and were not incorporated into the vulnerability matrices.

The consequences on residences of more conjunctive flood actions could be explored. Simultaneous actions have been explored through the f_{diff}-v combinations but while one residence wall experiences only the f_{diff}-v pressure differential, another wall could be experiencing waves or other hydrodynamic actions. Load transfers throughout the residence would need to be analysed. An example of a linear sequence of actions is a wall surviving an f_{diff}-v combination which nearly causes failure, but then the wall is no longer at its pre-flood strength when a piece of debris impacts. Similarly, saturated masonry or timber doors would have different strength characteristics than dry material. Therefore, predicting failure from a flood action following the initial inundation may not be the same as predicting failure from a flood action before the initial inundation. The most suitable approach to tackle multiple flood actions, conjunctively or in linear sequence, is likely to be a scenario basis.

Detailed analyses refining the vulnerability matrices would feed directly into developing an expanded flood failure flowchart. Such a diagram would assist in communicating all the

potential pathways by which residences could fail. This tool would also provide an analysis of the qualitative risks faced by residences from flood actions, so that those of most concern could be targeted for more detailed analyses including potential management options.

The expanded flood failure flowchart would help further quantitative analyses. Future research could assign probabilities to each failure pathway along with the cost of each outcome under different flood action scenarios. Quantitative risk could be calculated by selecting a flood scenario, creating a flood failure flowchart, and summing the probability of each pathway multiplied by the pathway's outcome. This procedure is a direct extension of the LEP method described in Section 13.2.

Another major task is developing vulnerability matrices and flood failure flowcharts for residences other than those observed in the case study sites. The techniques and methods developed in this dissertation could be easily applied to other locations and other residence types.

14.2.2 Applying the Vulnerability Matrices and Flood Failure Flowchart

The core research for this dissertation was developing the vulnerability matrices. Illustrative examples were provided of applying the matrices for risk calculations and for developing flood management options. This application could be extended further by choosing a management option, assuming that it is implemented successfully, and running LEP analyses for flood scenarios with that management option in place. If this procedure were followed for several management options over several different flood scenarios, then a comparison of the effect of different options could potentially be made without knowing the probability of the flood scenarios.

A powerful tool in decision-making would be available which eliminates much of the uncertainty associated with the probabilities of flood scenarios occurring. For example, if one management option consistently yields lower LEPs than another management option, this trend may be recognised and considered in selecting options. Alternatively, if trends do not emerge, then further analysis would be necessary, most prominently with respect to assigning probabilities to the flood scenarios. The ability to factor in probabilities for flood events would be an important extension to the applicability of the vulnerability matrices and the LEP method for quantifying risk.

One essential part of applying and analysing the tools developed in this dissertation is continual engagement of all sectors interested in flood management, including academia, the insurance industry and related sectors, government, and the public. The vulnerability matrices and flood failure flowchart are useful not only for estimating failures which could occur and the potential losses but also for communicating such issues to stakeholders. Communication and interaction amongst stakeholders is essential for producing the most appropriate, long-term solutions.

14.3 Final Words

This dissertation has illustrated that significant improvements could and should be made in the manner in which society manages natural disaster risk, especially regarding flood damage to modern residences in coastal, eastern England. The techniques developed can be expanded in a similar context beyond the work in this dissertation but could also be adapted to other flood situations, to other types of vulnerabilities, and to risks from other natural hazards. The tasks are complex, requiring a breadth and depth of knowledge which is likely to be beyond the capability of a lone researcher. Nonetheless, through collaboration and persistence, research and application related to reducing society's vulnerability to disasters will ultimately reduce losses and save lives.

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Table Ref.1: OS Historical Maps for Canvey Island

Date	Scale	Grid Code	Area
19th century to ≈1920 (County Series)	6" : mile	77SW,77SE 85NW,85NE	Nothing useful.
	25" : mile	77 #10,11,12,14,15,16	
		85 #2,3,5	
≈1920 to ≈1950 (County Series Amalgamated Edition) use Index 61		Essex 90	
after 1950ish (National Grid) inc. Digimap use Index Sheet 178	1:10,000 or 1:10,560	TQ78SE	
		TQ88SW	
	1:1250 with SW, SE, NW, and NE 1:2500 without SW, SE, NW, and NE	TQ7782,3,4 all	far W of island
		TQ7882,3,4 all	
		TQ7982,3,4 all	
		TQ8082,3,4 all	
		TQ8182,3,4 all	
		TQ8282,3,4 all	far E of island

Table Ref.2: OS Historical Maps for Hull

Date	Scale	Grid Code	Area
before ≈1950 (County Series)	6" : mile	for 1856: Yorkshire 240 Yorkshire 240NE	Wellington Street Area Victoria Dock Village
	25" : mile	239 #12	Hessle Foreshore
		240 #3,6,7	Wellington Street Area Victoria Dock Village
after ≈1950 (National Grid) inc. Digimap use Index Sheet 107	1:10,000 or 1:10,560	TA02NW	Hessle Foreshore
		TA02NE	Wellington Street Area
		TA12NW	Wellington Street Area Victoria Dock Village
	1:1250 with SW, SE, NW, and NE 1:2500 without SW, SE, NW, and NE	TA0225SW,SE	Hessle Foreshore
		TA0928SW,SE	Wellington Street Area
		TA1028SW	Wellington Street Area
		TA1028 TA1128	Victoria Dock Village

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