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THE AVOIDANCE OF PROGRESSIVE COLLAPSE: REGULATORY APPROACHES TO THE PROBLEM

E. F. P. Burnett

October 1975

Final Report

Report Prepared For: CENTER FOR BUILDING TECHNOLOGY
INSTITUTE FOR APPLIED TECHNOLOGY
NATIONAL BUREAU OF STANDARDS
WASHINGTON, D. C. 20234

Sponsored By: DIVISION OF ENERGY, BUILDING TECHNOLOGY AND STANDARDS
OFFICE OF POLICY DEVELOPMENT AND RESEARCH
DEPARTMENT OF HOUSING AND URBAN DEVELOPMENT
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The research and studies forming the basis for this report were conducted pursuant to a contract with the Department of Housing and Urban Development (HUD).

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PREFACE AND ACKNOWLEDGMENTS

In November, 1971, the U.S. Department of Housing and Urban Development (HUD) requested the National Bureau of Standards (NBS) to undertake a study of building safety with particular reference to the avoidance of progressive collapse. This project, being conducted by the Structures Section of the Center for Building Technology, was initially directed by Dr. Norman F. Somes and is currently under the direction of Dr. E.V. Leyendecker. Over the past three years, I have been privileged to be involved in the project, both as a guest worker at the NBS and as a consultant. In particular, I would like to acknowledge the assistance of Norman Somes and E.V. Leyendecker, both of whom have had considerable influence on this report.

To fully comprehend any building regulations is an achievement, to attempt to evaluate a foreign building regulation is a difficult, hazardous and somewhat presumptuous undertaking. Without the co-operation of numerous individuals and organizations this study would not have been possible. Two visits to Europe, during September and October of 1974, were necessary. In many instances individuals and organizations went to considerable lengths to ensure that my stay was both productive and enjoyable. Considerable correspondence was required. Interpretation, amplification and, often, the translation of the relevant building regulations was willingly provided. For this and for the many hours of discussion, I am indeed grateful. Whatever the merits of this study, the persons listed below deserve much of the credit; any shortcomings are solely the responsibility of the author.

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A special word of thanks is due to Paul Regan for his interest and assistance particularly with the Swedish and British regulations. Gerry Schorn's assistance with translating the relevant sections of the German regulations was greatly appreciated. Finally, my wife's forbearance and editorial assistance contributed greatly to the production of this study.

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CHAPTER 1: BUILDING SAFETY AND THE BUILDING DESIGN PROCESS

1.1 Introduction

Since the failure of the Ronan Point apartment building in London, England in 1968 most industrialized nations have been obliged to reconsider their regulatory conceptions of structural safety. The phenomenon of progressive collapse and how to accomodate those forms of loading not normally considered in design have been of primary concern. In particular, the structural implications of an unexpected explosion or accidental impact have had to be evaluated. In some countries where the problem was of immediate and vital concern, separate interim provisions or guidelines to supplement existing building codes and specifications were issued. The repercussions of these criteria had, in at least one case, a significant effect on the building industry while the question of how to avoid progressive collapse has continued to be a controversial issue within the design profession.

Recent studies [1.1, 1.2, 1.3] suggest that in the United States the frequency and severity with which abnormal loadings such as gas or bomb explosions occur is relatively significant. During the last few years there have also been a number of well-publicized building failures. Moveover in at least two sets of design recommendations, namely the Operation Breakthrough Guide Criteria (Volume 1) and the HUD draft document on "Criteria for Structural Design of Buildings to Avoid Progressive Collapse", specific design provisions are recommended. In each case the provisions have been based largely on the British recommendations prior to release of their new unified Code CP 110. In August, 1973, an amendment was made to the City of New York Building Code that specifically required the avoidance of progressive collapse. It is evident that some form of regulatory action at a national or industry-wide level is urgently needed. There is understandable concern among the various sectors of the industry that any action taken should be professionally, structurally and economically viable.

Several countries have recently issued new or updated versions of their relevant building codes. These take into account the statistics, the research, the many technical developments and the experience that has accumulated since 1968. In particular, the revised codes of the United Kingdom, Canada, Germany and the Scandinavian countries attempt in different ways to address the problem. Various international organisations such as the Comité Européen du Béton (C.E.B.), the International Council for Building Research (C.I.B.) and the Nordic Concrete Association (N.E.U.) have studied and elaborated upon this topic. Revised recommendations must reflect the experience of the six years since Ronan Point. These revisions presumably consolidate the positive and eliminate or moderate the contentious aspects of previous provisions. There is obviously much to be gained from the experience of others even if only in avoiding their mistakes. An up-to-date summary and synthesis of foreign codes and design recommendations should be of value to those responsible for formulating regulations and the design profession in general. This study is therefore an attempt to review, compare, and evaluate those regulatory provisions that are directed at abnormal loadings and the avoidance of progressive collapse. The relevant codes of practice or building regulations of all those countries that have attempted to address this aspect of structural safety will be considered.

Of course the issue of avoiding progressive collapse is only part of the much larger problem of ensuring structural integrity. While safety may be the main concern, the problem must be viewed initially in the broadest possible context. To sustain this generality of perspective it is necessary to develop a consistent and systematic basis for discussion. Accordingly the nature and mechanics of the structural design process will be considered.

1.2 The Structural Design Process

The structural design process encompasses both conception and execution. The computational aspect of the latter can be considered to be the systematic satisfaction of a series of performance criteria whereby the structure, at the material, section, element, sub-system and system levels of response, is required to perform in a manner that is better than or at least consistent with that required by the client or user or the relevant building authority. This process may be represented as follows:

$$\Sigma_c \Sigma_w [R^B_{c,w} \geq R^P_{c,w}] \begin{matrix} \beta_{max} \\ \$_{min} \end{matrix}$$

where the term

- R^B represents the behavioral response of the structure (as represented by the mathematical models and assumptions permitted by the relevant codes)
- R^P represents the required performance normally specified by the relevant building authority or client or user
- β represents benefit
- \$ represents cost
- and the subscript c represents the structural criterion involved. These criteria may be classified as either serviceability (e.g., crack width, deflection) or safety (e.g., flexural resistance) criteria. Σ_c indicates that all criteria must be individually considered
- w represents the loading situation under consideration. Each and every relevant loading level and combination must be considered, hence Σ_w.

Thus, for each criterion this performance equation must be satisfied for all pertinent levels and combinations of loading within the general framework of minimum cost and maximum benefit. While it may be relatively simple to define the process, design is in reality a complex, multi-stage, and iterative procedure. For example, quantifying benefit is difficult enough without attempting simultaneously to maximize benefit and minimize cost. To reduce all significant structural design requirements to algebraic criteria is not always possible and sometimes unnecessary. Moreover, there are many problems associated with the actual loadings; for example, their relative significance, their nature and their magnitude.

This study has some bearing on all of the parameters involved in the performance equation and it is therefore instructive to discuss each of these parameters in relation to abnormal loadings and the avoidance of progressive collapse.

1.2.1 Structural Safety (c,β,$)

Most structural design criteria may be classed as either serviceability or safety related. This study is primarily concerned with considerations of building safety.* There are at least three aspects of safety that should have some influence on structural design;

i) life safety: human injury and death

ii) economic safety: property damage, the short-term costs (shoring, repair, loss of income, etc.) and the long-term costs (demolition, investment considerations, reputation, etc.) thereof

iii) social safety: both the short-term (disruption, loss of shelter) and the long-term (relocation, re-zoning) consequences.

To some extent these aspects can be quantified and all have a significant influence on the cost-benefit trade-off. In North American building design practice, it is customary to reduce any explicit consideration of structural safety to the simple requirement that the estimated "ultimate" load carrying capacity of the structural system (i.e., W_u) be greater than or at least equal to the code specified overload (i.e., W_o). This oversimplified deterministic requirement can be expressed as follows:

$$R^B_{s,w_o} \geq R^P_{s,w_o}$$

---

* This is perhaps unduly restrictive since abnormal loadings may be of consequence insofar as serviceability is concerned, e.g., accidental impact in parking structures. The performance of buildings with regard to abnormal forms of loading could therefore be viewed in a much wider context. An additional limitation is that this study is largely concerned with structural considerations. It should however be borne in mind, especially for abnormal forms of loading, that the form, appearance and utilization of a building also have an interactive influence on the demands imposed upon and the behavioral response of the structure of a building.

where s indicates that the above is a system level strength criterion and w_o indicates that the load level concerned is the specified overload. Simpler still, this performance criterion is, for design purposes, usually reduced to:

$$W_u \geq W_o$$

For this criterion it is evident that:

i) limited explicit consideration can be given either to benefit or cost,

ii) risk is not involved although, presumably, the value of the overload factors specified in the codes do somehow provide for the probability of application of individual loads or critical combinations thereof,

iii) considerations of risk or benefit are largely obviated by ensuring that provided the actual loads are similar to and never exceed the normally specified overloads, structural failure cannot occur.

There are obviously good reasons for re-examining existing considerations of design safety.

1.2.2 Loading (W)

Recent events and currently available studies [1.1, 1.2, 1.3] suggest that the various forms of loading specified in existing North American Codes of practice may not necessarily be the most significant. Moreover, the coverage and detail provided by these codes may be inadequate especially where structural safety is concerned. At a more fundamental level there is some doubt as to which loads are covered in most building regulations.

A recent study by the author [1.4] attempted to develop a comprehensive classification system for all forms of loading that may influence a completed building. This is summarized in Table 1.1 and Figure 1.1. Relative to current North American practice it is possible

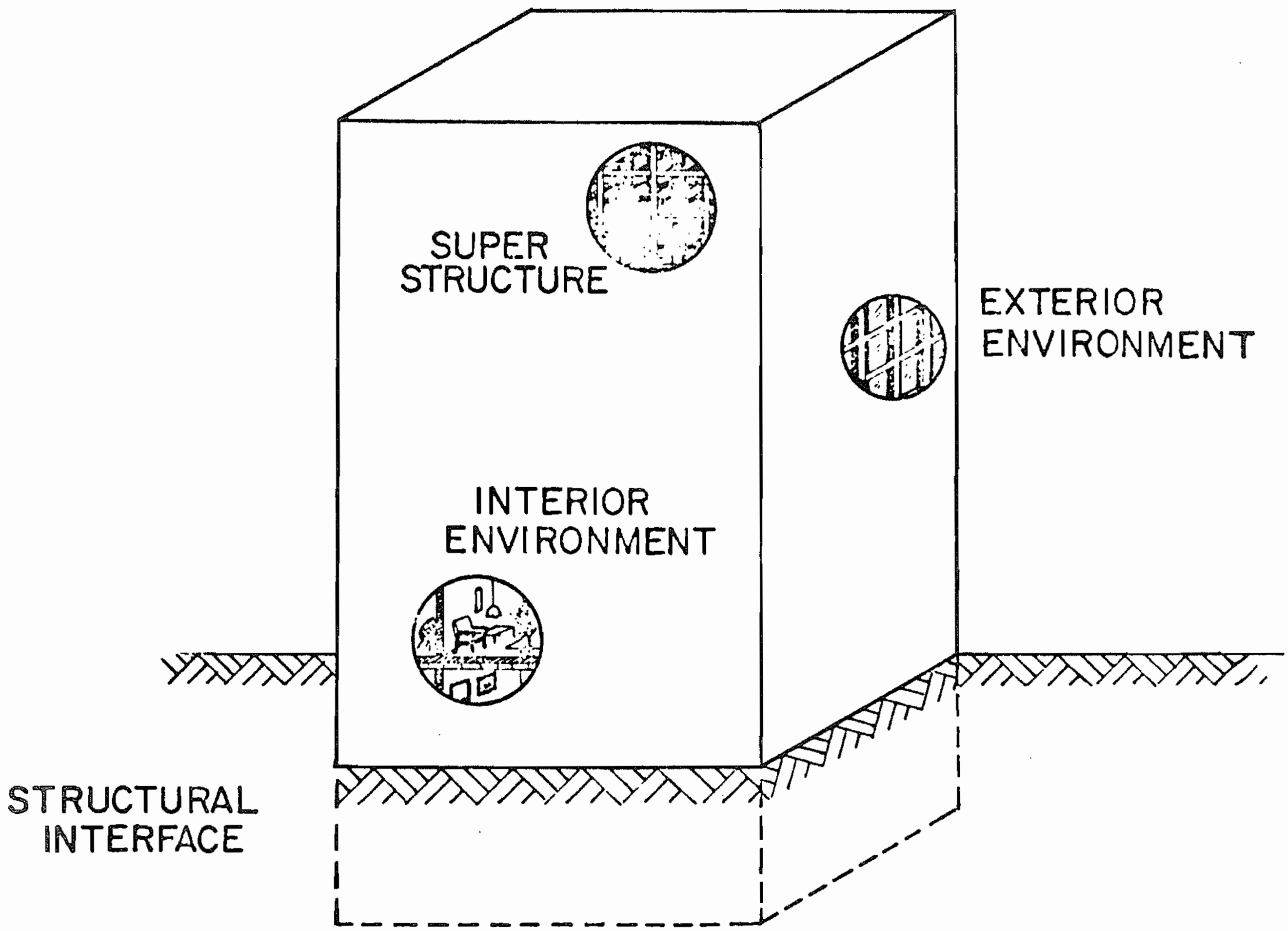

Fig. 1.1 REPRESENTATIVE BUILDING

CATEGORY OF LOADING	SOURCE OR CAUSE	TYPE: STATIC	TYPE: DYNAMIC	TYPE: IMPACTIVE	NATURE AND RELEVANCE: SERVICE: DETERMINISTIC	SERVICE: PROBA-BALISTIC: SIGNAL	SERVICE: PROBA-BALISTIC: INCIDENCE	SAFETY: DETERMINISTIC	SAFETY: PROBA-BALISTIC: SIGNAL	SAFETY: PROBA-BALISTIC: INCIDENCE	REMARKS
A. SUPERSTRUCTURE	1 NON-VARIABLE [1] GRAVITY LOADING	A			⬤				•		"DEAD" LOADS
	2. VARIABLE [1] GRAVITY LOADING (i) OCCUPANT	A				⬤			⬤	•	TRAFFIC AND OCCUPANTS
	(ii) SUPERIMPOSED	A				⬤			⬤		MOVABLE PARTITIONS, FURNITURE, EQUIPMENT ETC.
	(iii) EQUIPMENT [2]		A		⬤				•		EQUIPMENT IN OPERATION e.g. MOTORS, ELEVATORS
	(iv) ACCIDENTIAL	A		A						•	DEBRIS
	3. RHEOLOGICAL CONSIDERATIONS	R			⬤						SHRINKAGE, POSSIBLY CREEP
	4. "LACK OF FIT" CONSIDERATIONS	D							•	•	ACCUMULATED TOLERANCES, VARIABILITY OF QUALITY ETC.
B. EXTERIOR ENVIRONMENT	1. WIND		A			⬤	•		•	⬤	
	2. WATER (i) DIRECT	A			⬤					•	PONDING ON ROOFS ETC.
	(ii) CONSEQUENTIAL	D			⬤				•		LEAKAGE, CONDENSATION
	3. SNOW	A				⬤			⬤	•	
	4. WEATHERING	D			⬤			⬤			INCLUDES LONG TERM FREEZE - THAW EFFECTS
	5. THERMAL	R			⬤				•	•	PRIMARILY SHORT TERM VOLUME CHANGE (INCLUDING FIRE)
	6. POST - FIRE	D						⬤			AS A CONSEQUENCE OF FIRE e.g. RESIDUAL STRESSES, DEMAGE ETC.
	7. OVERPRESSURE (i) SONIC BOOM		A				•		⬤	•	
	(ii) EXPLOSION		A						•	⬤	BOMB, HAZARDOUS MATERIALS IN TRANSIT, GAS
	8. COLLISION (i) VEHICULAR			A		⬤			•	⬤	ALL GROUND TRANSPORTATION VEHICLES
	(ii) AIRCRAFT			A						⬤	CIVILIAN OR MILITARY AIRCRAFT INCLUDING HELICOPTERS
	(iii) OTHER			A						⬤	A CONSTRUCTION INCIDENT, CRANES ETC.
C. INTERIOR ENVIRONMENT	1. SERVICE SYSTEM (i) EXPLOSION		A						⬤	•	GAS OR STEAM EXPLOSIONS
	(ii) FLOODING	A								•	WATER, OIL ETC.
	2. THERMAL	R				⬤			⬤	•	PRIMARILY FIRE
	3. POST-FIRE	D						⬤			
	4. PRESSURE VARIATION (i) STACK EFFECT	A			⬤						
	(ii) EXPLOSION		A						⬤		BOMBING, GASOLINE, PROPANE CYLINDER OR COMBUSTION EXPLOSION
	5. WEAR	D			⬤			•			
D STRUCTURAL INTERFACE	1. SEISMIC ACTION		R			⬤			•	⬤	EARTH TREMORS QUAKES AND BLASTING EFFECTS
	2 GROUND MOTION		R			⬤					DUE TO UNDERGROUND TRAFFIC, PILING, ETC.
	3 FLOODING	A								•	
	4. SETTLEMENT	R			⬤			⬤	•		DUE TO SCOURING ETC

LEGEND :
- A ACTIVE TYPE OF LOADING
- R REACTIVE TYPE OF LOADING
- D DETRACTIVE TYPE OF LOADING
- ⬤ OF PRIMARY CONCERN
- • OF SECONDARY CONCERN

NOTES.

1. NON-VARIABLE IN THE SENSE THAT ANY CHANGE, IF MADE, WILL BE MADE UNDER THE SUPERVISION OF A STRUCTURAL ENGINEER. VARIABLE MAY REFER TO EITHER TEMPORARY STATIC LOADS E.G. MOVABLE PARTITIONS, OR TO THE NON-STATIC LOADS GENERATED BY MECHANICAL OR ELECTRICAL EQUIPMENT IN OPERATION.
2. THE DEAD LOAD OR SELF WEIGHT IS INCLUDED AS A NON-VARIABLE GRAVITY LOAD.
3. ONLY THOSE PHENOMENA THAT MIGHT AFFECT THE COMPLETED BUILDING ARE CONSIDERED.

TABLE 1.1 STRUCTURALLY SIGNIFICANT LOADING PHENOMENA to group these loadings as follows:

i) Those loadings that existing codes and standards require to be considered in the design process either directly or indirectly.

ii) Those loadings that could be considered but in practice are usually neglected, minimized, eliminated, or only implicitly included in design.

iii) Those loadings for which there are no explicit or implicit design requirements and which are rarely, if ever, considered in design. These forms of loading are:

Variable gravity loading: accidental
Overpressure: sonic boom
Overpressure: explosion
Collision: vehicular
Collision: aircraft
Collision: other
Service system malfunction: explosion
Service system malfunction: flooding
Pressure variation: explosion
Flooding

This last group of loadings is of particular interest in that they are all the consequence of either an accident or some abnormal situation. They will collectively be referred to as abnormal loadings* and they may be re-grouped by type, as follows:

* The phrase "abnormal loading" appears to be the least misleading collective title. It has a dual meaning in the sense that (i) these loads are not normally considered and, (ii) they may also be accidental or extraordinary or localized or excessive or extreme.

1. Pressure loading resulting from:
   i) an explosion e.g., related to the service system (natural gas, steam, etc.), stored gas (butane, propane, oxygen etc.), stored liquid (gasoline, liquefied natural gas), hazardous material in transit, or bombing due to civil or criminal action, or combustion.
   ii) wind-induced localized overpressures due to a tornado, hurricane, etc.
2. Impact loading due to:
   i) ground vehicle collision
   ii) aircraft collision
   iii) missile, e.g.: wind or water-borne object (debris), military weapon, failure of adjacent building, construction accident, falling debris caused by some internal structural failure.
3. Static loading due to:
   i) water or other service system malfunction;
   ii) debris resulting from some incident, e.g., flooding, that is structurally significant.

A series of studies on the incidence and consequence of the following forms of abnormal loading have been completed and are currently available:

Gas service system explosion [1.5]
Bomb explosion [1.6]
Vehicular collision [1.7]
Aircraft collision [1.8]
Sonic boom [1.9]
Hazardous materials in transit [1.10]

For the U.S. it has been demonstrated [1.1] that at least the first three phenomena occur often enough with structural consequences severe enough to warrant serious consideration. There are of course problems in that the loads or representative levels of load have to be defined and that considerations of risk, even relative risk, are difficult to resolve. Moreover, our conception of safety in design is rather limited especially where explosive or impactive loadings are concerned. As a first step, it is evident that some knowledge of how other countries view abnormal forms of loading would be helpful.

1.2.3 Structural Response (R^B, \$)

Given that gas or bomb explosions or vehicular collision are (individually or collectively) structurally significant forms of loading, it will be necessary for the designer to have (i) some knowledge of the actual response of the structure to these loadings, (ii) an acceptable behavioral model and (iii) a design methodology. Some level of damage is usually involved and clearly the building should be able to accommodate reasonably severe but representative values of the loading concerned.

For economic reasons the behavioral model should not be overly conservative; for example, a simple elastic approach would be both economically expensive and behaviorally unrealistic. A failure theory involving large deformations and possibly permitting membrane and/or catenary action is probably required. Current U.S. building design practice with regard to failure theories, design methods or detailing procedures, makes little or no provision for explosive or impactive loading. Changing current practice to take into account such loadings would have widespread repercussions on both the design process and construction practice. It is, therefore, very important that before any regulatory changes are made serious consideration be given to experiences elsewhere. Considerations of response are also of importance because it is probably in this area where the most research and development will be required.

1.2.4 Structural Performance (R^P)

The simplistic strength safety criterion that the structure at least be able to resist the maximum anticipated loading, i.e., $W_u \geq W_o$, merely reduces the performance aspect, i.e., the right hand side of the inequality, to a specified series of overload values. With explosive or impactive loading and, for that matter, seismic or wind loading, the performance demanded of the structure should take into account the following factors:

i) Short term considerations, i.e., during and immediately after application of the critical loading and prior to failure. These concern the avoidance of a brittle or instantaneous failure, the extent and nature of the indications or warning signs of impending failure, the period of time required to evacuate the building, the extent of initial damage, etc.

ii) Longer term considerations, i.e., post incident but prior to repair or demolition. These involve the performance of the damaged structure, i.e., the loads that may have to be sustained and the stress levels that may be permitted.

To provide explicitly for all or even some of these considerations would require significant changes to current building design practice. In any attempt to establish priorities and precedents, to quantify the relevant requirements and then to develop acceptable code clauses, a study of the comparable situation outside the United States should be of considerable assistance.

1.3 Objectives and Scope of Study

The primary objective of this study is, therefore, to examine the regulatory situation in those countries where sustained consideration has been given to abnormal loadings and building safety. Those measures taken to avoid the phenomenon of progressive collapse are of special interest. In each case the pertinent sections of the latest building code or regulations will be considered. The impact of these regulations on the building industry, especially on the structural design process and on the building itself, i.e., its configuration and structure, will be evaluated. Where possible some background to the formulation of the regulations as well as likely developments will be provided.

While the perspective is necessarily and unavoidably North American, every attempt will be made to evaluate relevant regulations within the prevailing economic, social and technical context of the country concerned. This, of course, is easier said than done. Because the Western European countries and Canada do have much in common with the U.S., this study will largely be devoted to these countries.

In order to provide constructive input to the development of regulations in the U.S. an attempt will be made to synthesize the approaches and provisions studied. A flow-chart will be developed to indicate all possible regulatory options. An attempt will also be made to develop values for the forces, regions of acceptable damage, overpressure values, etc., that represent an acceptable consensus of the regulations studied. Consideration will also be given to problems faced by the structural designer.

This study is mainly concerned with structural concrete, in particular precast panelized construction. This does not mean that structural steel or timber or masonry buildings are not subjected to abnormal loadings or are invulnerable to progressive collapse. Indeed a comparable study for masonry buildings, both clay and concrete masonry, would be of considerable value.[1.14] The scope of this study is therefore somewhat limited but nonetheless does have some relevance for materials and buildings other than those involving concrete. It should be noted that less detailed but similar studies have been performed by Lewicki and Olesen [1.11], Granstrom and Carlsson [1.12] and Pume and Witzany [1.13].

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CHAPTER 2: UNITED KINGDOM

2.1 Introduction

Since the Ronan Point collapse on May 16, 1968, numerous amendments and revisions to the various building regulations and recommended codes of practice have been made. The more significant documents as far as structural design and progressive collapse are concerned, are listed chronologically in Table 2.1.

DATE OF ISSUE	TITLE OF DOCUMENT
October, 1968	"Report of the Inquiry into the Collapse of Flats at Ronan Point, Canning Town", Griffiths, H., Pugsley, A., and Saunders, O. [2.1].
November, 1968	Circular No. 62/68 Ministry of Housing and Local Government [2.2].
April, 1970	Fifth Amendment (1970) [2.3] to the Building Regulations (1965).
1970	London Building (Constructional) Amending By-Laws 1970 [2.4] and the Related Notes for Guidance (prepared in consultation with the District Surveyors' Association) [2.5].
1970	Addendum No. 1 (1970) [2.6] to British Standard Code of Practice CP116 (1965) and CP116: Part 2 (1969) [2.7].
January, 1971	"The Resistance of Building to Accidental Damage", Statement RP/68/05 by the Institution of Structural Engineers [2.8].
1971	Seventh Amendment (1971) [2.9] to the Building Regulations (1965).
June, 1972	The Building Regulations (1972) [2.10].
November, 1972	British Standard Code of Practice CP110 (1972) "The Structural Use of Concrete" [2.11] and the Related Handbook Produced by the Cement and Concrete Association [2.12].

Table 2.1 - List of Significant Documents Subsequent to the Ronan Point Failure

In addition there exists a plethora of documentation on progressive collapse, abnormal loadings and building safety. This is not surprising given the economic and social impact of Ronan Point; contributing factors have been the number of building authorities, the multiplicity of relevant codes and the active involvement of various non-governmental organizations such as the Institution of Structural Engineers (I.S.E.), the Cement and Concrete Association (C. & C.A.), the Brick Development Association and the Construction Industry Research and Information Association (C.I.R.I.A.). Probably the most significant document has been the Fifth Amendment [2.3] which has now been incorporated in The Building Regulations (1972) [2.10] as regulations D.19, D.20 and D.21. The best guide to the regulations and their structural implications is probably the London Building (Constructional) Amending By-Laws 1970: Notes for Guidance [2.5]. The development of the Unified Code CP110 [2.11] is particularly significant since it will probably replace the three codes CP114 [2.13], CP115 [2.14] and CP116 [2.7] and be "deemed to satisfy" the majority of Building Authorities.* CP110 attempts to be completely general in its treatment of structural concrete and clearly is a re-evaluation and consolidation of the events, regulations, controversies and experience of the four years after Ronan Point. Summaries, both technical and historical, of the events, regulations and design recommendations prior to the release of CP110 [2.15 - 2.19] have been published. This study therefore will be restricted to an evaluation of CP110 insofar as the sections pertaining to progressive collapse or abnormal loadings influence structural design and building practice.

* The main obstacle to immediate acceptance appears to be the fact that CP110, unlike the other codes, is expressed in limit state terms. At present CP110 and CP114, CP115 and CP116 have dual validity.

2.2 CP110: Relevant Code Clauses Plus Comment

2.2.1 Philosophy

The basic philosophy of the unified code is expressed under the heading "Design: Objectives and General Recommendations", as follows:

2.2.2 Ultimate limit state. The strength of the structure should be sufficient to withstand the design loads taking due account of the possibility of overturning or buckling. The design strengths of materials and the design loads should be those defined in **2.3**, as appropriate for the ultimate limit state. This assessment should ensure that no ultimate limit state is reached as a result of rupture of one or more critical sections, by overturning or by buckling caused by elastic or plastic instability, having due regard to the effects of sway when appropriate.

The layout of the structure on plan, and the interaction between the structural members, should be such as to ensure a robust and stable design: the structure should be designed to support loads caused by normal function, but there should be a reasonable probability that it will not collapse catastrophically under the effect of misuse or accident. No structure can be expected to be resistant to the excessive loads or forces that could arise due to an extreme cause, but it should not be damaged to an extent disproportionate to the original cause.

In addition, due to the nature of a particular occupancy or use of a structure (e.g. flour mill, chemical plant etc.), it may be necessary in the design concept or a design reappraisal to consider the effect of a particular hazard and to ensure that, in the event of an accident, there is an acceptable probability of the structure remaining after the event, even if in a damaged condition.

Significantly CP110 acknowledges the existence of normal and abnormal situations and the fact that, in the latter case, absolute safety may not be possible. The second paragraph is particularly interesting in that terms such as "normal function" and "reasonable probability" are used. While neither "reasonable probability" nor the criterion "that damage should not be disproportionate to the original cause" are quantified, the intent and scope of the code is clear.

Section 2.3 of CP110 specifies values for the partial safety factors associated with the normal applied loads and the material properties when considering the effect of excessive loading or localized damage, i.e., 1.05 for the loads and 1.3 and 1.0 for the concrete and steel respectively.

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* These sections of CP110 are reproduced with the permission of the British Standards Institution.

2.2.2 *Continuity Criteria*

These requirements are contained in a section of CP110 under the heading "Design and detailing: reinforced concrete" and are as follows:

3.1.2.2 ***Stability.*** To accord with the criteria of **2.2.2** the layout of the structure in plan and the interaction between the structural members should be such as to ensure a robust and stable design. It is recommended that:

(1) in the design for ultimate wind loads the horizontal force taken into account be not less than $1\frac{1}{2}$ % of the total characteristic dead load above any level; this force may be shared by the parts of the structure depending on stiffness and strength;

(2) to obviate the possibility of vehicles running into and damaging or removing vital loadbearing members of the structure in the ground floor, the provision of bollards, walls retaining earth banks, etc., be considered;

(3) in buildings of five or more storeys an effectively continuous vertical tie be provided from foundation to roof level in all columns and walls. The area of this tie should be at least equal to the minima given in **3.11.4.1** for their main reinforcement, but see **5.1.2.4** for walls designed as plain concrete.

(4) all buildings be provided with effective horizontal ties round the periphery and internally. At re-entrant corners or at substantial changes in construction care should be taken to ensure that the ties are adequately anchored or otherwise made effective.

Preferably, the ties should be so placed as to provide the best assistance in resisting, by cantilever, catenary or other actions the results of extreme damage by accidental causes. The horizontal forces to be resisted by the ties should be derived from *a*, *b* and *c* below using as the value of F_t the lesser of the values obtained from

$F_t = (20 + 4n_s)$ where n_s is the number of storeys in the building, or

$F_t = 60$

In providing the ties in *a*, *b* and *c*, it may be assumed that no other forces are acting and that the reinforcement is acting at its characteristic strength. Reinforcement provided for other purposes may be regarded as forming a part of, or the whole of, these ties.

a. Peripheral tie. At each floor and roof level an effectively uninterrupted peripheral tie should be provided capable of resisting a tensile force of F_t kN, located within 1.2 m of the edge of the building or in the perimeter wall.

b. Internal ties. In addition to the peripheral tie, internal ties should be provided at each floor level in two directions approximately at right angles. The internal ties should be effectively uninterrupted throughout their length and should, unless they continue as column or wall ties (see *c*), be anchored to the peripheral tie at both ends. The ties should be capable of resisting a tensile force in each direction respectively of

$$F_t \frac{(g_k + q_k)}{7.5} \frac{l}{5} \quad \text{kN per metre width}$$

but not less than F_t kN per metre width;

---

* Characteristic strength means the yield or proof stress of the reinforcement.

where $(g_k + q_k)$ is the sum of the average characteristic dead and imposed loads per unit area of the floor in kN/m^2,

l is either the greatest distance in metres in the direction of the tie under consideration between the centres of the columns or other vertical load bearing members whether this distance is spanned by a single slab or by a system of beams and slabs, or

5 times the clear storey height (under beams, if any), whichever be the lesser.

Where the vertical loadbearing members are walls which on plan occur in one direction only (e.g. cross wall or spine wall construction) then the ties provided parallel to the walls should be capable of resisting a force of F_t kN per metre width.

Part or all of the internal ties may be spread evenly over the width of the structure or may be grouped at beams, walls or other appropriate intervals. The ties may be in the floor slab, in beams or in walls; where they are in a wall they should be located within 0.5 m of the top or bottom of the floor slab.

c. *Column and wall ties.* Each external column and every metre length of external wall should be anchored or tied horizontally into the structure at each floor level with a tie capable of developing a force equal to the greater of:

(i) $2F_t$ kN, or $\left(\frac{l_o}{2.5}\right) F_t$ kN, whichever be the lesser, where l_o is the floor to ceiling height in metres.

(ii) 3 % of the total ultimate vertical load at that floor level for which that member has been designed.

Corner columns should be tied into the structure at each floor level in each of two directions approximately at right angles, with ties capable of developing a force equal to the greater of (i) or (ii) above.

Column and wall ties may be partly or wholly the same reinforcement as that provided for the peripheral or internal ties.

These stability or, more precisely, continuity criteria, do not require much elaboration. Apart from the references to minimum lateral design loads and the desirability of avoiding vehicular impact, Section 3.1.2.2 is primarily concerned with the provision of minimum tie requirements. The following comments are pertinent.

2.2.2.1 Vertical Ties

Without stating why five stories should be so significant, sub-clause (3) specifies that in buildings of five or more stories the following minimum tie reinforcement must be provided:

3.11.4.1 ***Minimum area of main reinforcement.*** The area of tension reinforcement in a beam or slab should not be less than 0.15 % $b_t d$ when using high yield reinforcement, or 0.25 % $b_t d$ when mild steel reinforcement is used, where b_t is the breadth of the section and d is the effective depth. For a box, T- or I-section, b_t should be taken as the average breadth of the concrete below the upper flange.

The minimum number of longitudinal bars provided in a column should be four in rectangular columns and six in circular columns and their size should not be less than 12 mm. Except for lightly loaded columns (see 3.5.1.1) the total cross-sectional area of these bars should not normally be less than 1 % of the cross-section of the column.

A wall cannot be considered as a reinforced concrete wall unless the percentage of vertical reinforcement provided is at least 0.4 %. This vertical reinforcement may be in one or two layers.
It should be noted that for fire resistance purposes, a wall containing less than 1.0 % of vertical reinforcement is classed as a plain concrete wall.

Thus for cast-in-place reinforced concrete construction, regardless of the height of the building, normal good practice should suffice for vertical continuity. These requirements are of practical consequence for plain concrete and, although not clearly stated, for buildings with precast components. In both of these cases the necessity for vertical continuity applies only to buildings of five or more stories but the provisions of code section 5 (see Section 2.2.3 of this report) must also be satisfied.

One criticism is the choice and use of the five story criterion in Section 3. It is clearly unnecessary for cast-in-situ construction. It perhaps would have been more appropriate to introduce this criterion in Section 5, if indeed there is any need to introduce it at all. In any event CP110 implies that in buildings (and by default this can only apply to the plain concrete or precast parts thereof) of less than five stories vertical continuity is unnecessary.

2.2.2.2 Horizontal Ties

Compared to the provisions for vertical ties those for horizontal ties are much more precise and comprehensive. Firstly, the minimum magnitude of the required tie force rather than a minimum amount of steel is specified. Secondly, horizontal tie requirements, unlike vertical tie requirements, apply also to buildings of less than five stories. Thirdly, the magnitude of the horizontal tie force reflects the fact that the probability of occurrence of an abnormal loading increases with building height, see Figure 2.1. Fourthly, some provision is made for the influence on risk of variations in the magnitude of the dead and superimposed load, floor span and floor-to-ceiling height.

One omission is that, while a peripheral tie is required at all floors and the roof, the internal tie is only required at each

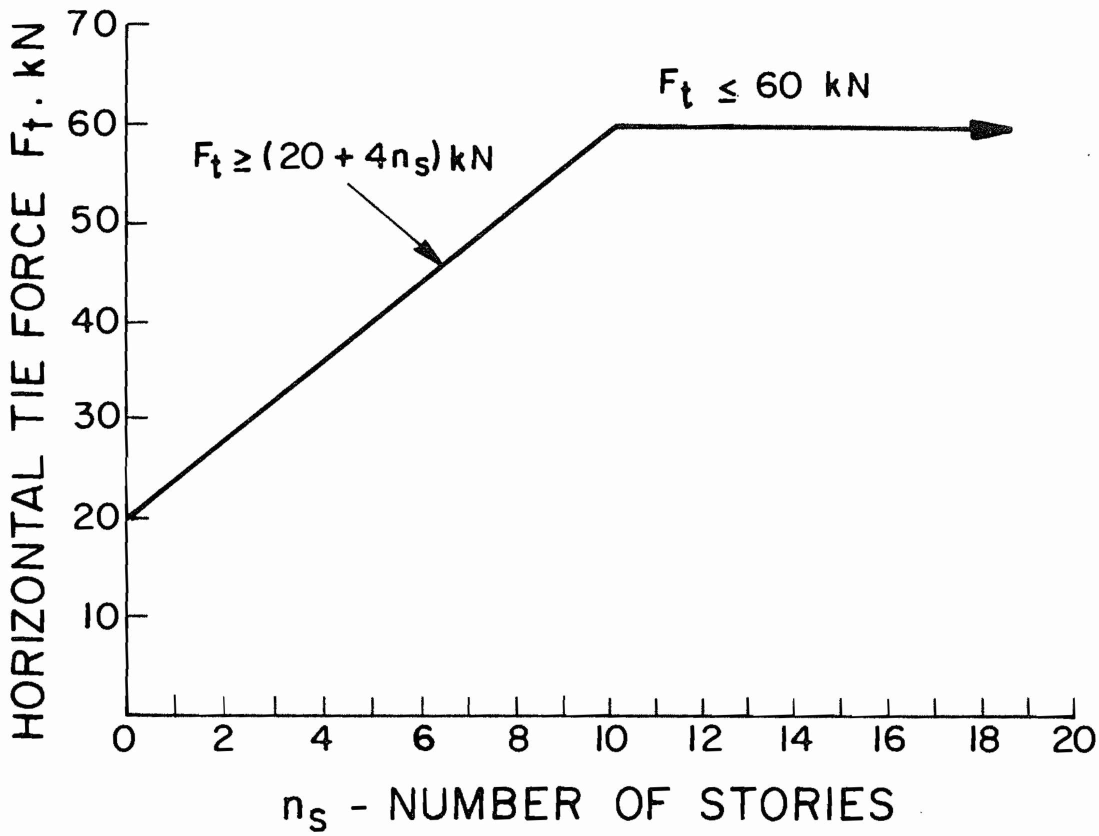


Fig. 2.1 HORIZONTAL TIE FORCE v STORY HEIGHT OF BUILDING (CP 110)

floor. Apparently, CP110 does not require internal ties at roof level even though roof members may be more vulnerable than floors since they are more susceptible to weathering, temperature and, possibly, abuse.*

Table 2.2 provides some evolutionary background for these values while Table 2.3 summarizes sub-clause (4) and attempts to illustrate, by way of an example, the relative impact of compliance with these provisions. It is evident that for the average cast-in-place R.C. building these tie forces are quite nominal and do not add to the reinforcement content although they may affect the detailing [2.15]. However, these explicit continuity requirements do have a significant impact on buildings involving precast or plain concrete elements. In this case, the additional provisions of Sections 5.1.1 and 5.1.2.4 also apply.

It should be noted (see note a. in Table 2.2) that the tie force values given in CP110 are limiting or "ultimate" values whereas in previous documents a service load force was specified. The service load force of 1700 pounds per foot used in both Addendum No. 1 [2.6] and the I.S.E. document RP/68/05 [2.8] is based on a beam or floor span not exceeding 17 feet (5 m) and on a gross weight of floor and imposed loads not exceeding 150 p.s.f. (7.2 kN/m^2). The magnitude and derivation of the various tie forces will be further discussed in Section 2.2.3 but note that CP110 requires internal tie forces to be increased for spans in excess of 5 m and total loads in excess of 7.5 kN/m^2. This is achieved by the expression

$$\left[\frac{(g_k + q_k)}{7.5} \cdot \frac{\ell}{5}\right]$$

or, in more familar units and terminology, $\left[\frac{(w_D + w_L)}{150} \cdot \frac{\ell}{17}\right]$.

* A recent letter from Mr. Eric Bunn indicated that amendments to CP 110 are about to be issued in order to remedy this omission.

Table 2.2 - Comparison of Minimum Horizontal 'Tie-Forces' [2.15]

No. of Stories	Peripheral Tie, kN [K]			Internal Ties, kN/m [K/FT.]			Column Tie kN or Wall Tie kN/m		
	CP110	Addendum No. 1[a]	ISE's[a] RP/68/05	CP110	Addendum No. 1	ISE's RP/68/05	CP110	Addendum No. 1	ISE's RP/68/05
0	20 (4.5)[b]			20 (1.37)			20		
1	24 (5.4)	36 (8.1) (can be part of internal tie)	none	24 (1.64)	22.75 (1.6) (main tie)	none	24	22.75	none
2	28 (6.3)			28 (1.92)			28		
3	32 (7.2)			32 (2.19)	11.4 (0.8) (transverse)		32		
4	36 (8.1)			36 (2.47)			36		
5	40 (9.0)			40 (2.74)	45.5 (3.10) (main tie)		40		
6	44 (9.9)			44 (3.01)	22.75 (1.6) (transverse)		44		
7	48 (10.8)			48 (3.29)	or 34.5 (2.4) in two directions		48		
8	52 (11.7)	73 (16.4) (can be part of internal tie)	none	52 (3.56)		45.5 (3.12) in two directions	52	45.5 (at top of wall)	none
9	56 (12.6)			56 (3.84)			56		
10	60 (13.5)			60 (4.11)			60	22.75 (at bottom of wall)	
over 10	60 (13.5)			60 (4.11)			60		

Note: a. The forces given in the Addendum No.1 [2.6] and in the Institution of Structural Engineers paper RP/68/05 [2.8] have been converted into 'ultimate' terms for comparison with CP 110, e.g., working load of 1700 lbf per ft (25 kN per metre) with permissible stress of 0.55 f_y when converted 25/0.55 = 45.5 kN per metre.

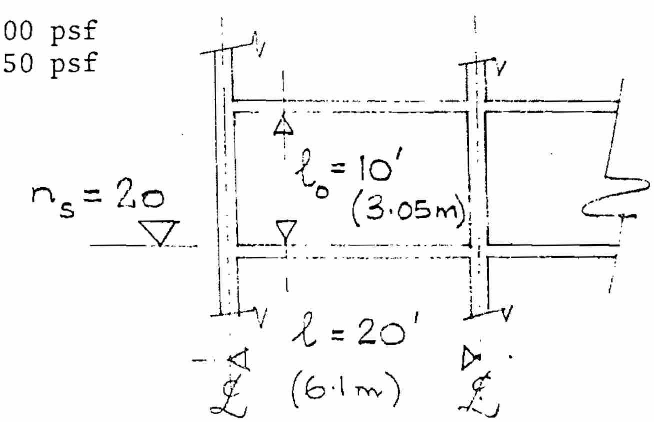
b. Values in kips or kips per foot are shown thus (4.5).

Table 2.3 - Horizontal Tie Requirements.— CP 110 Section 3.1.2.2

	S.I. Units		P. F. S. Units		A_s^* - Amount of reinforcement
	Tie Force Expression	Example	Tie Force Expression	Example	
F_t	$(20+4n_s) \leq 60$ kN.	60 kN	$(4.5+.9n_s) \leq 13.5$k.	13.5k.	-
Peripheral Tie Force	F_t kN	60 kN	F_t k	13.5k.	2#3
Internal Tie Force	$F_t \frac{(g_k+q_k)}{7.5} (\frac{\ell}{5}) \geq F_t$ kN/m for q_k, g_k in kN/m^2	70 kN/m	$F_t \cdot \frac{(g_k+q_k)}{156.7} (\frac{\ell}{16.4}) \geq F_t$ k/f for q_k, g_k in psf.	4.8 k/f	#3at 16 1/2"c/c
External Column and Wall (per metre length) Tie Force	$F_t \, (\frac{\ell_o}{2.5}) \leq 2F_t$ kN	73 kN	$F_t \, (\frac{\ell_o}{8.2}) \leq 2F_t$ K	16.5 k (or 5 k/f of wall)	3#3

Example Structure:

Dead Load = 100 psf
Live Load = 50 psf



*For 60 grade reinforcing steel the allowable tie force per
#3 bar = 0.11 x 60 = 6.6 kips.
#4 bar = 0.20 x 60 = 12 kips.

2.2.3 *Precast, Composite and Plain Concrete Construction*

Design and detailing provisions pertaining to structural safety for these forms of construction are given in Section 5.1 of CP110.

5.1.1 Scope of Section 5. This Section is concerned with the additional considerations which arise in design and detailing when precast members or precast components including large panels are incorporated into a structure or when a structure in its entirety is of precast concrete construction. It also covers the use of plain concrete for walls.

5.1.2.4 ***Stability.*** The recommendations regarding stability given in **3.1.2.2** apply also to precast, composite and plain concrete construction.

The tie forces referred to in **3.1.2.2** should normally be provided by reinforcement or prestressing tendons embedded in precast or in situ structural elements.

Ties should generally be joined by one of the methods described in **5.3.2, 5.3.3** or **5.3.4** except that simple lapped joints should only be used when their adequacy has been proved by relevant tests.

Ties connecting precast panels should be so arranged as to minimize out of balance effects.

Ties should only be located in the joints between precast panels if those joints are of sufficient size and detail to transmit the forces from the reinforcement into the precast units and to develop the required strength at any lapped joints.

Column and wall ties should not, for their anchorage at either end, rely solely on the bond of a straight bar. Bars should be bent or hooked so as to provide the required anchorage in bearing on sound concrete unless welded or mechanically anchored to the main reinforcement in a precast member.

For a normal building which is five or more storeys in height where the vertical tying together does not comply with (3) of **3.1.2.2** or where in a building supported by plain concrete walls the area of the effective vertical ties from foundation to roof level is less than 0·2 % of the cross-sectional area of the walls or where all the precast floor and roof units are not effectively anchored in the direction of their span either to each other over a support or directly to their supports in such a manner as to be capable of resisting a horizontal tensile force of F_t kN per metre width:

(1) the structure should be so designed that, at each storey in turn, any single vertical loadbearing element (other than one complying with (2)) can become incapable of carrying its load, without causing collapse of the structure or any significant portion of the structure. In designing the structure for this condition, account may be taken of any building components which are otherwise non-loadbearing. When reliance is placed on catenary action, allowance should be made for the horizontal reactions necessary for equilibrium.

In the case of a wall, the length considered to be a single loadbearing element should be taken as the length between adjacent lateral supports or between a lateral support and a free edge. For the purposes of this definition of wall length only, a lateral support may be considered to occur at

a. a stiffened section of the wall (not exceeding 1 m in length) capable of resisting a horizontal force of $1.5F_t$ kN per metre height of wall, or

b. a substantial partition at right angles to the wall, provided that it is tied to the wall with a tie force equal to $0.5F_t$ kN per metre height of wall (a substantial partition may be taken as one having an average weight of not less than 150 kg/m^2).

except that

(2) any vertical loadbearing element, which cannot be allowed to become ineffective, may be designed, together with its connections, to withstand a load of 34 kN/m^2 applied to it from any direction. Any horizontal member, or part of a horizontal member, which provides lateral support vital to the stability of that vertical loadbearing element must be designed, together with its connections, to withstand a load of 34 kN/m^2 applied to it from any direction. Any member or lateral support so designed must also be capable of supporting the reaction from any attached building components also subject to a loading of 34 kN/m^2 or such reaction as might reasonably be transmitted having regard to the strength of the attached component and the strength of its connection.

To accord with **2.3.3.1** and **2.3.3.2,** when a structure is designed in accordance with (1) or a vertical loadbearing element is designed in accordance with (2) the partial safety factor for loads (γ_f) should be taken as 1.05 and the partial safety factor for strength (γ_m) should be taken as 1.3 for concrete and 1.0 for steel.

The initial clauses of Section 5.1 relate to tie placement, lapping and anchorage and this aspect will be discussed later in this study. Of principal interest are the two alternate design procedures that must be applied to all those buildings

i) that involve plain concrete walls where there is either no continuous vertical tie or where the amount of vertical tie steel is less than 0.2 per cent of the area of the related wall section.

ii) that involve precast concrete components or composite construction and are five or more stories high where either the vertical tie is not effectively continuous or the amount of tie reinforcement is less than specified in 3.11.4.1.

iii) where all the precast floor and roof units are not effectively anchored (i.e., the horizontal tie force is less than F_t per unit length) onto or across their perimeter or support members.

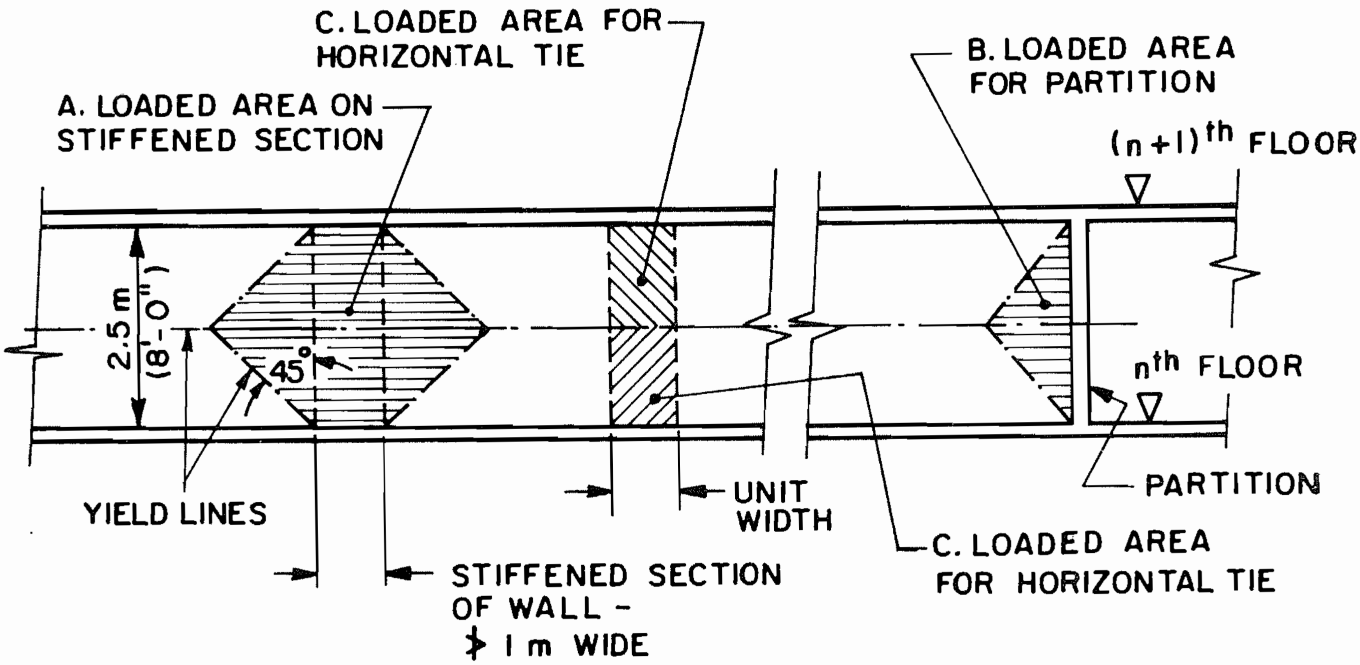
Before discussing the two alternate design procedures the following comment regarding vertical ties may be appropriate.

In general, the language and style of CP110 is admirably precise but the seventh paragraph of Section 5.1.2.4 is ambiguous and the above interpretation complies with that in the Handbook [2.12]. It is evident that the five story or more criterion only applies to precast or composite construction. No reason is given why there should not be any vertical tie requirement for precast buildings of less than five stories nor is any argument concerning the level of risk put forward. It is quite possible that a good risk-benefit case could be made for ignoring low-rise buildings. On the other hand, there are various reasons, structural, professional and/or economic, why the low-rise residential situation could be considered to be significant. For example, any three-story walk-up apartment building assembled from precast floor units and load bearing masonry walls that is serviced with gas is both prone and vulnerable to abnormal loading. Specifically to exclude any vertical tie requirement from this type of building would appear to be a serious omission.

The two design procedures namely (1) the alternative path method and (2) the equivalent static pressure approach, are alternatives only as far as the individual elements are concerned. All vertical load bearing elements must comply with at least one of these procedures with the result that, insofar as the overall building is concerned, the check on structural safety is really a two stage process whereby

i) all vertical load bearing elements are each in turn notionally removed. If significant or overall failure, i.e., collapse, were to occur then procedure (2) must be adopted. In the Fifth Amendment [2.3] it was necessary to limit structural damage following the removal of a member to the story above and the story below that in which the incident occurred, and horizontally localized to 750 square feet or 15 per cent of the relevant floor area. It is worth noting that CP110 does not attempt to define a permissible damage volume or quantify the term 'significant portion of the structure'. In applying method (1) much is left to the discretion of the designer and, as far as behavioral response is concerned, the designer is explicitly permitted to utilize catenary action.

ii) in the event that there are any single vertical load bearing elements whose removal would result in extensive structural failure then these elements, their boundaries and all interdependent elements must be designed to accommodate a static pressure of 34 kN/m^2 (5 psi). Although in CP110 there is no explicit reference to a gas explosion this loading is intended to be equivalent to a severe gas explosion and was formally introduced in November, 1968 [2.2]. There has been considerable debate concerning the choice of this model; for example, an explosion is a dynamic load, no provision for venting has been made, and the magnitude of 5 psi could be overly conservative. The magnitude and nature of this particular forcing function has probably been the single most contentious issue in the drafting and subsequent application of the British regulation. It is significant that in spite of all the debate and criticism the 5 psi criterion was retained in CP110. In fact, its influence is much greater than would appear merely from the wording of paragraph 5.1.2.4.(2). Consider, for instance, a multistory building with a floor to ceiling story height of 2.5 m (8 ft.). If an explosion were to occur between floors, equivalent horizontal forces would be generated at the wall to floor joints, at transverse partitions and over the vertical walls. As shown in Figure 2.2, these forces have a direct relationship with the horizontal tie force provisions of CP110. It is worth emphasizing the fact that for a 5 psi loading and an 8'-0" high vertical wall, the equivalent service level wall-floor force equals 1584 pounds per foot (i.e., 720 x 4 x 1 x 55%). This equivalent load is comparable to the 1700 pound per foot tie force specified in previous regulations.



A. Stiffened Section

Average force $\doteq$ (1.0+1.25)34 = 76.5 kN/m
Per unit height (5.3 k.per foot)
of wall

Refer to CP110 Section 5.1.2.4.(1) a. which specifies an average force of 1.5 F_t for $F_t \leq 60$ kN/m, 1.5 $F_t \leq 90$ kN/m

B. Partition

Average force $\doteq$ (0.625x34) = 21.25 kN/m
per unit height (1.5 k.per foot)
of wall

Refer to CP110 Section 5.1.2.4.(1) b. which specifies a tie force of 0.5F_t for $F_t \leq 60$ kN/m, 0.5 $F_t \leq 30$ kN/m.

C. Horizontal Tie (Wall to Floor)

Force $\doteq$ 1.25x34 = 42.5 kN/m
(2.9 k.per foot)

This force is comparable to the internal horizontal tie force required by CP110 in a six story building (See Table 2.2) Also note that the 1700 pounds per foot value (See Table 2.2a) in "ultimate" terms is equivalent to 3.1 k.per foot

Figure 2.2 - Effect of Equivalent Explosion (34 kN/m^2 or 5 psi) Within Building. (After [2.15]).

There is another approach to the evaluation of the internal tie forces which can be derived from the "alternative path approach" i.e., design procedure (1), rather than the equivalent static pressure approach. This procedure is documented in a Technical Instruction issued by the Ministry of Public Building and Works* [2.20]. The approach is illustrated in Figure 2.3 and is based on the assumption that catenary action is developed when a support is removed such that vertical equilibrium is maintained. For the loads and dimensions assumed in Figure 2.3, it is evident that the resultant tie forces are equal to the maximum values required by CP110. A further point to note is that certain dimensions and loads namely 8'-0" (2.5 m) clear height, 17'-0" (5 m) span, and 150 psf (7.2 or 7.5 kN/m^2), crop up fairly regularly in the British literature.

2.3 Detailing

Provided that the structural member does not have to be designed to accommodate an overpressure of 5 psi, it would appear that the provisions of CP110, in terms of quantities of reinforcement, are not particularly onerous. Of real significance, especially for precast or plain concrete elements, is the distribution and nature of the reinforcement, i.e., the detailing. In particular, considerable attention is directed at the inter-connection of structural components.

For example, sub-sections 3.1.2.2 (3) and (4) call for effectively continuous ties in the vertical and all horizontal directions. In providing these ties it may be assumed that no force other than the specified tie force is acting and the reinforcement operates at its characteristic strength, i.e., f_y. As the reinforcement provided for other purposes may be regarded as forming either a part or the whole of the tie it will be found that for most structures the flexural reinforcement provided for the usual dead, imposed and wind loads will, with minor additions and/or modifications, fulfil these tie requirements [2.21].

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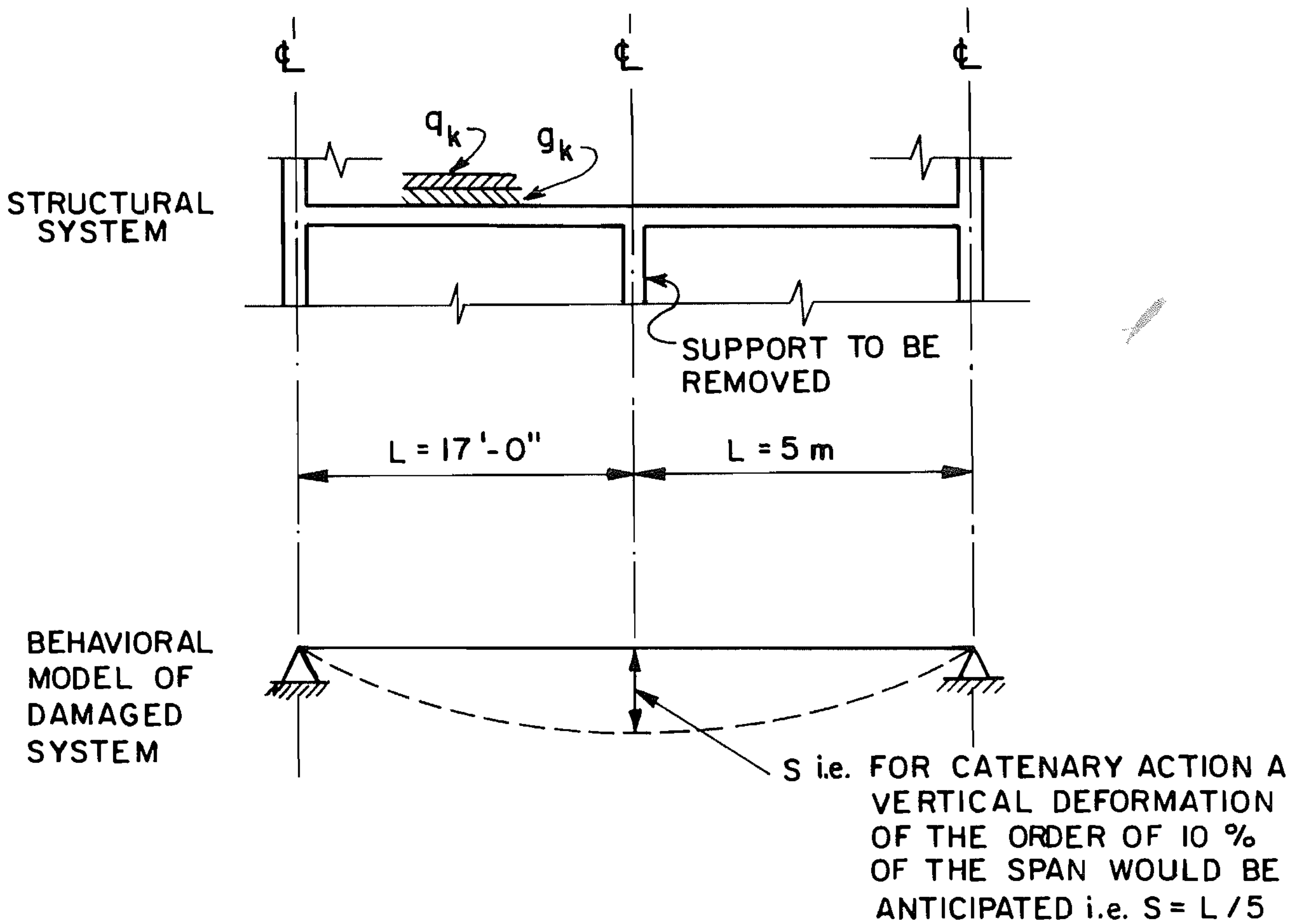
* Currently the Department of the Environment.



For unit width of floor,
For full dead load (g_k) and 1/3 live load ($q_k/3$),
and moment equilibrium:

$$\frac{(g_k + q_{k/3})}{8}(2L)^2 = F_t\, S$$

$$\therefore F_t = \frac{(g_k + q_{k/3})L^2}{2S}$$

for $L = 17'\text{-}0''$

$S = L/5$

$g_k = 75$ psf ($3.6\ kN/m^2$)

$q_k = 75$ psf ($3.6\ kN/m^2$)

$\therefore F_t = 4.25$ k/Ft (60 kN/m)

Figure 2.3 - Catenary Model for Evaluation of Horizontal Internal Tie Force

It is therefore suggested [2.21] that the structure be designed for the normal loadings and then checked to ensure both sufficiency and continuity of reinforcement. Not only must the ties be effectively uninterrupted, i.e., connected, within and across all structural elements but in certain circumstances (e.g., 3.1.2.2.b) the ties in different directions must be anchored together, e.g., the internal tie must always be anchored to the horizontal peripheral ties. These requirements affect detailing because

i) the tie steel in one direction will often be at a different level or at an inconvenient location relative to other tie reinforcement. According to Somerville [2.21] it is not permissible to vary levels e.g., to go from top to bottom bars, if the tie is to be continuously effective.

ii) maximum prefabrication of reinforcement cages is usually desirable.

It is, of course, implicit that careful attention is paid to splice and development length, i.e., bond provisions. Significantly, the requirements of Section 3.11.6, "Bond, Anchorage and Bearing" appear to be more stringent than current North American provisions and considerably more stringent than the more readily comparable requirements of ACI 318-63 or the 1970 National Building Code of Canada.

Some detailing requirements are illustrated in Figures 2.4, 2.5 and 2.6. Figure 2.4 shows methods of anchoring internal and peripheral ties; Figure 2.5 shows the junction of peripheral ties at a corner column while Figure 2.6 demonstrates one method of ensuring an effectively continuous horizontal tie. Both directly and indirectly the stability provisions of CP110 and before that the Fifth Amendment [2.3] have had a considerable effect on detailing practice. For a more comprehensive overview of this impact reference should be made to the publication, "Designed and Detailed [CP110: 1972]" by Higgins and Hollington [2.22].

Most of the examples quoted above apply specifically to cast-in-place construction. The problem is obviously compounded in the case of precast construction. The difference between pre- and post-Ronan

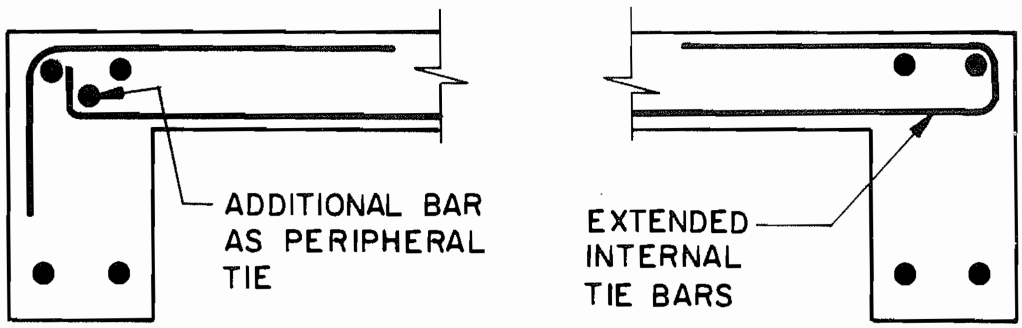


Fig. 2.4 : EFFECTIVE ANCHORAGE OF INTERNAL TO PERIPHERAL TIES.

Point detailing is perhaps exemplified by Figure 2.7 and Figure 2.8 which illustrates the vertical and horizontal tie situation. In addition, internal ties are required at each floor level in two directions at right angles (sub-section 3.1.2.2.b) which means that with precast floor panels effectively continuous ties must be provided in the transverse direction. Little imagination is required to appreciate that the adoption in North America of CP110's tie requirement would have considerable repercussions with probably greatest impact on three story walk-up residential construction where very little if any provision for behavioral continuity is made (except in earthquake zones). Merely to emphasize the importance of detailing where abnormal loadings are concerned, reference should be made to the paper by Rhodes [2.23] which summarizes his experience in monitoring bomb explosive damage to buildings in Northern Ireland.

2.4 Concluding Comment

Undoubtedly the issues of accommodating abnormal loadings and/or the avoidance of progressive collapse have had considerable impact on the building industry in the United Kingdom. This discussion of CP110 may give some idea of the problems faced by the design profession but does not, nor was it intended to, cover the economic or social consequences of Ronan Point. In order to obtain a measure of the possible direction and nature of future developments some of the current R and D activities in the U.K. should be mentioned.

In particular, reference should be made to the final report [2.24] of the Structural Stability Panel of the Construction and Housing Research Advisory Panel (CHRAC) which indicated that the three main areas requiring research were:

i) the incidence, consequences and risk associated with explosives. Most importantly incident surveys were undertaken with the intention of also evaluating structural response to abnormal loadings;

ii) the performance of structural joints; and

iii) structural analysis with particular consideration for structural safety.

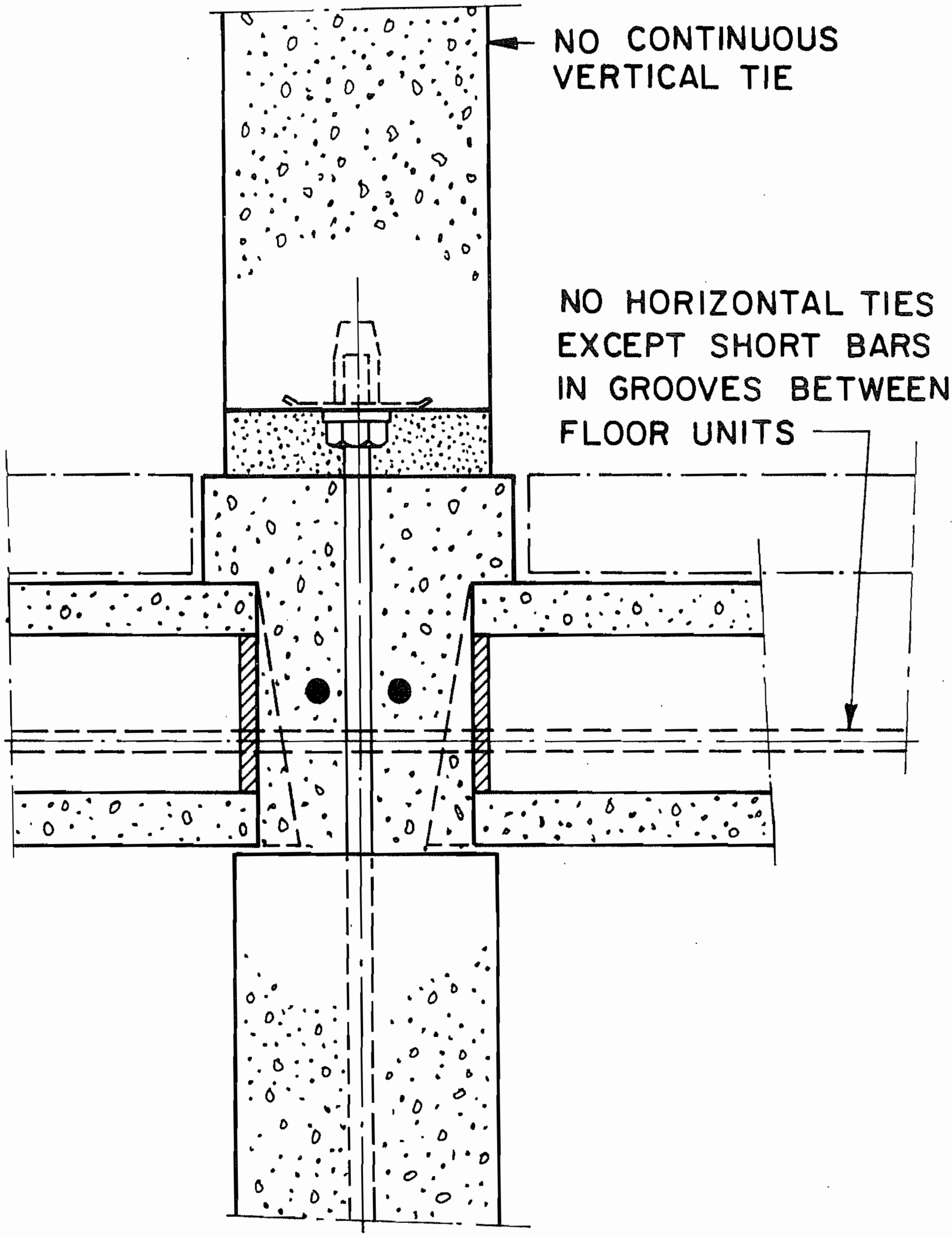


Fig. 2.7 : REPRESENTATIVE PRE-RONAN POINT JOINT DETAIL [2.16]

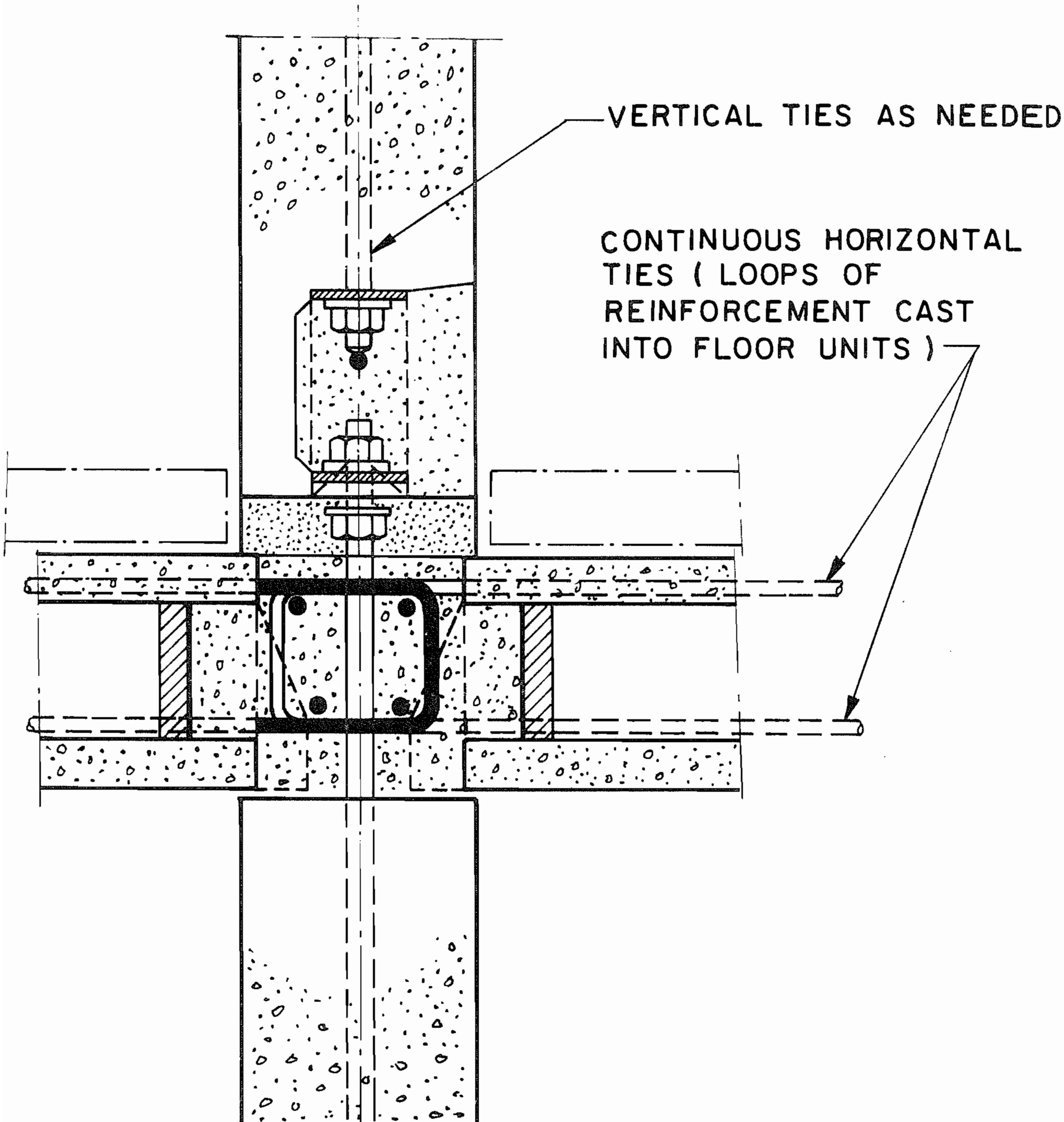


Fig. 2·8 : REPRESENTATIVE POST-RONAN POINT JOINT DETAIL [2.16]

That this committee, since October 1970, has directly authorized expenditure on research of approximately $180,000 and has given its support to the expenditure of a further $450,000 is some indication of the continuing importance of these aspects of structural integrity. Gifford [2.18] has provided a list of a number of the major studies done to date. Special mention should also be made of the R and D activities involving masonry. The research results and latest draft of the new Masonry Code are of particular interest to North America and for completeness the latter should have been included in this study.

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CHAPTER 3: SWEDEN

3.1 Introduction

The current nationally valid building regulations, Svensk Byggnorm 67, SBN 67 [3.1] have been in effect since January 1, 1968*. Supplementary regulations to deal with abnormal loadings, and progressive collapse were introduced on July 1, 1973 [3.3]. This document, SBN 22:35, supplements Chapter 21, "Load Conditions", and Chapter 22, "General Demands on Loadbearing Building Components" and comprises mandatory regulations, advisory clauses and commentary. Translations [3.3A, 3.3B] exist and a complete English text is incorporated in this report.

A companion or explanatory document [3.4]** has been drafted to expand upon SBN 22:35 and provide, by means of illustration and calculation, specific design guidelines. Sune Granstrom and Martin Carlson who were responsible for writing this document, have also compiled a comprehensive survey of loadings and their effect on building safety [3.5]. In this study various failure theories and related experimental work are reviewed.

The Swedish Building Regulations are currently being revised and are due for re-issue in mid-1975. The 1973 supplement SBN 22:35 has been incorporated into the draft of the new Building Regulation and this draft, [3.6] is being reviewed. As the new version exists only in draft form, discussion will largely be restricted to the current regulations.

From a professional standpoint Sweden has possibly the most comprehensive regulations dealing with abnormal loading and related problems. In fact, the general topic of structural safety with regard to abnormal loading has been under close and consistent study since 1970. This is reflected in the nature of the regulations as well as

* For a comprehensive discussion (in English) of the regulatory aspects of the Swedish Building Control System, see reference [3.2].

**The author is indebted to Dr. Paul Regan for a translation of this document.

the existence of extensive documentation [3.4, 3.5].

An English translation of the regulatory and the advisory clauses in SBN 22:35 follows, while a translation of the commentary to these clauses is provided in Appendix S at the end of this Chapter.

3.2 Translation* of the Supplements** (Dated July 1, 1973 to Chapters 21 and 22 of the Svensk Byggnorm SBN 22:35 Statens Planverk, Publication No. 63, Fortskridanda Ras (Progressive Collapse)

21:93 Exceptional, Unpredictable, Additional and/or Accidental Loads and Effects

Buildings shall be designed and built with regard to the risk of progressive collapse as a consequence of local damage caused by exceptional, unpredictable additional and/or accidental loads and effects (see 22:352 and 22:353); exceptions are made however for one-family houses and for buildings where the risk of casualties caused by progressive collapse is low.

:931 Below are listed some examples of "exceptional, unpredictable and/or accidental loads and effects that are not normally accounted for in design".***

a) Explosions of town gas, natural gas, volatile liquids, solvents, dust-air mixtures or explosives.

b) Impact by road vehicle, aircraft, ship, machines, etc., blows from swinging or falling loads, e.g., from building cranes, factory cranes, etc. The magnitude of impact will be greater than that ordinarily covered in design specifications.

c) Unforeseen settlements causing the structure to behave in a manner not envisaged in normal design.

d) Overloading due, for example, to carelessness.

e) Unforeseen weakening of individual building components caused by fire.

General regulations for the design of buildings with regard to excessive loadings are given in SBN 22:35.

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* This translation is a composite of that made and provided by Paul Regan [3.3B] and that prepared by Statens Planverk [3.3A]. Alterations were made by the author largely to avoid inconsistencies.

**Mandatory Regulations are in italics.

***The collective phrase "excessive loadings" will be used instead of "abnormal loadings" in the remainder of the translation. This is in deference to Dr. Granstrom who regards this term as being closest to the intent of the Swedish regulations.

22:35 Design for the Prevention of Progressive Collapse

:350 Introduction

:3501 References

All Swedish Building Standards (SBN) and their contents are listed in the Register of Swedish Building Standards (SBN-R). For the significance of regulations and advisory notes, see Section 0:11 of SBN 67. For "Excessive Loadings" see SBN 21:93 above.

:3502 Definitions

By "progressive collapse" is meant a structural failure that occurs when localized primary damage in a building leads to severe damage of parts of the building other than in the immediate vicinity of the region of primary damage.

"Primary damage" is damage produced directly by the "excessive loadings", e.g., the hole made by a vehicle breaking or pushing out parts of a building. In the event of an explosion, the primary damage is considered to be the structural damage that occurs in the building during the first fraction of a second due to the pressure and shock waves produced by the explosion. When failure occurs in a floor or a wall, the primary damage is assumed to include the damage due to debris down to the next floor below.

By the "region of primary damage" is meant the volume or area that originally contained the components constituting the primary damage.

"Severe damage" is taken as involving damage or dislocation sufficient to occasion serious risk of human injury. Cracks and local deformations are not viewed as severe damage in this context.

By the "region in the immediate vicinity of the region of primary damage" is meant a volume or area, the extent of which is related to the volume or area of the region of primary damage. Thus, for example, if the region of primary damage were a room unit or its equivalent, the immediate vicinity could comprise one or two or possibly three or four similar units.

An "overall stabilizing function" is the action of a building component ensuring the stability of a building as a whole.

The "main loadbearing direction" (after damage) of a structural element is to be taken as the shortest span for a slab, the span for a beam, the longitudinal direction for a column, and the in-plane horizontal direction for a wall.

:351 General

The risk of progressive collapse due to excessive loadings shall be accommodated either by taking measures to limit the extent of failure in the event of local damage in accordance with 22:352, or by reducing the level of risk in accordance with 22:353.

:352 Measures for Limiting Collapse as a Consequence of Local Damage

Buildings shall be designed so that local primary damage, which may be caused by excessive loading in any part of a building, shall not produce severe damage other than in the region of primary damage and its immediate vicinity.

:3521 Buildings with loadbearing walls and of height not exceeding five times the breadth of the building can be assumed to satisfy the requirement of limiting collapse, if the two principles of 22:35211 and 22:35212 are applied simultaneously.

:35211 With regard to a building as a whole the overall stabilizing function shall be distributed so that local damage due to excessive loading will not endanger the overall stability of the building. The adequacy of this distribution shall be assessed on the assumption of a cubic volume of damage with a side length equal to the greatest of the following dimensions:

1 story height including two floors
1/10th of the height of the building
1/20th of the length of the building

The cubic volume of damage shall be assumed to be positioned in the most unfavorable way, but with its sides parallel to the floors and walls of the building.

With this volume assumed to be damaged, the building shall be able to resist at least half the design wind load, acting simultaneously with the other loads. The stresses in the stabilizing components may correspond to yielding (or its equivalent).

:35212 With regard to those parts of the building structure whose failure, separation or removal could cause progressive collapse, it shall be ensured that local damage can be bridged over by membrane action in the floors and beams or by cantilever action of the walls, and that satisfactory connections exist through the components and across the joints between them. The necessary connection can be assumed to be achieved if the following tensile forces can be resisted at every section through such components or across the joints between them.

Buildings with four stories or less

The requirements for connections applies only to horizontal tensile forces and not to the vertical direction. Each such section (or joint) shall be able to resist a force of at least

20 KN per metre length of section (or joint). In the main loadbearing direction the sectional resistance (or joint resistance) shall also be at least equal to the self weight of the component in question, and for floor slabs at least equal to the self weight plus the permanent live load according to 21:3. For walls the load calculation is to be based on a length equal to twice the story height.

Buildings with more than four stories, but not more than eight stories

The minimum requirement is similar to that for buildings with four stories or less. However, the requirement of resistance in the main loadbearing direction is increased by 10 per cent for each per story in excess of four stories, and this increase is to be applied to all the stories of the building. In addition vertical ties (horizontal sections and corresponding joints) are required in all exterior loadbearing components, i.e., within or connected to the facades. The vertical resistance shall be at least 20 KN per metre length of section (or joint).

Buildings with more than eight stories

Connection or tie requirements are similar to the above except that vertical ties are required in all vertical loadbearing components and not only in facade walls. However, for buildings of more than 16 stories, the additional requirement for connections in the main loadbearing direction shall be decided for each individual case in consultation with the relevant authorities.

Irrespective of the number of stories, tie requirements are to be considered to be fulfilled only if all joints and structural components outside of the primary damage region possess sufficient ductility to function properly and resist specified forces even in the event of large deformations caused by localized damage.

:3522 With the special permission of the relevant authorities, means other than those given in 22:3521 may be used to satisfy the requirement of limiting the extent of damage due to an excessive loading. In such cases, the results of investigations or tests shall be presented to show that either the proposed type of construction does not have a greater tendency to progressive collapse than one designed according to 22:3521, or the probability of progressive collapse is especially low for some reason.

:353 Measures for Reducing the Risk of Damage Due to Excessive Loading

The building shall be designed and constructed in such a way that it can satisfactorily withstand every conceivable excessive loading or effect.

:3531 This provision can be met either by giving the building such strength and form as to reduce its vulnerability or by lowering the risk. The latter can be done by reducing the probability of occurrence of an excessive loading to a sufficiently low level.

3.3 Discussion

The extensive commentary that accompanies the regulations in SBN 22:35 has been translated and is included as Appendix S to this report. This discussion is intended to supplement the commentary and has been limited to analyzing the nature and evaluating the impact of the Swedish regulations.

The regulations approach the whole question of abnormal loading and related building safety in a very direct manner; the risk is acknowledged, the various types of loading phenomena are enumerated, the structural generality of the problem is emphasized, and a specific methodology for structural design is suggested. Indeed, the regulations are remarkably concise and rational.

The design alternatives are clear cut. Option 1: Limit the extent of damage (i.e., avoid overall failure) when an abnormal loading occurs or, option 2: Reduce risk to a satisfactory level. Acceptable levels of risk or even relative risk are never stated but the regulations do recognize that for small buildings (e.g., single family dwellings) or low-occupancy buildings (e.g., transformer buildings), considerations of progressive collapse are meaningless.

A possible criticism is that the emphasis here appears to be restricted to the safety of the building occupants. If structural safety is considered also to encompass property damage and the economic and social consequences of structural failure, then consideration of abnormal loadings in small or low-occupancy buildings may be necessary. While the problem may have to be solved in non-structural terms e.g., by revising regulations for the distribution of gas, some action may be necessary to lower risk (i.e., the second design option).

The mandatory regulatory clauses (i.e., 21:93, 22:351, 22:352, and 22:353) are completely general in that all building types and all forms of abnormal loading are considered. The companion document [3.4] is also very general in terms of its coverage and intent. However, Clauses 22:3521, 22:35211 and 22:35212 virtually limit the application of the first design option to load bearing wall structural systems with a height of not greater than 16 stories or five times the breadth of the building. This would appear to apply to most multi-story buildings in Sweden.

3.3.1 Option 1

Clause 22:35211 accomplishes the following:

i) suggests that the stabilizing elements of a building, e.g., cores, shafts, etc., should be distributed throughout the building. This conceptual aspect is particularly important when planning the layout of the building and is expanded upon in [3.4].

ii) quantifies what is meant by damage volume. This is the structural volume that is assumed to be incapable of taking load, i.e., effectively removed. At the very least the volume is a cube of side equal to the story height plus the thickness of one floor. Apparently [3.7] the concept is also intended to

- quantify the requirement that stabilizing structural elements be advantageously distributed throughout the building.
- provide some idea of possible debris loading although there is no mandatory requirement to accommodate loads of this nature.

This volumetric definition does acknowledge the three-dimensional nature of the consequences of phenomena such as an explosion and may avoid some of the ambiguities inherent in the one-element "damage" definition used in, for example, the British Code CP110. The volume involved is a cube of side at least equal to the story height plus the thickness of one floor element. This damaged or notionally removed volume is much greater than that in any other regulation and can be a particularly demanding criterion. To illustrate this aspect consider the following examples:

a) the situation at the ground floor of a large multi-story office building where the actual story height may be of the order 20 to 30 feet. In North America there would typically be a single central core region which would almost certainly be in structural difficulties if a 20 to 30 foot damage volume were to be removed.

b) Since the minimum dimension is specifically one story height plus the floor depth, this means that not only the basic element but also all the related joints could be removed. As illustrated in Figure 3.1, the removal of a cube of side (h + t) from a multi-story, multi-bay structure effectively eliminates the continuity of all members across the joints involved and for all loadbearing purposes a structural volume much greater than $(h + t)^3$ should be considered to have been damaged or removed.

These criticisms may not be warranted in a Swedish context in view of the differences between North American and Swedish building practice. One intended use of the damage volume concept is to provide guidance in planning the location of laterally stiff cores or sections in a building. Further reference should be made to [3.4] for guidance as to the use of this concept.

Clause 22:35212 complements 22:35211 and is primarily concerned with the interconnection of elements rather than the overall structure. Clause 22:35212 accomplishes the following:

i) quantifies minimum tie force values: see Table 3.1 and Figure 3.2. These values reflect the variability of risk with building height as well as with the relative importance of different elements.

ii) the tie forces to be provided are numerically small and even for a 16 story building entail only small quantities of reinforcement. It must, however, be emphasized that these tie forces are to be accommodated across all horizontal joints. For example, a precast concrete floor slab would have to be effectively tied across all four edges. There are various ways of accomplishing these connections and one way, observed on a construction site, is illustrated in Figure 3.3. Figures 3.4 and 3.5 respectively show details of novel lateral and support connections for precast concrete floor slabs. It is evident that in terms of labor, supervision, complexity and thus time (but not necessarily materials), the provision of ties will involve some additional cost.

iii) specifies that tie provisions also extend to buildings with less than four stories. In North America, a comparable regulation would

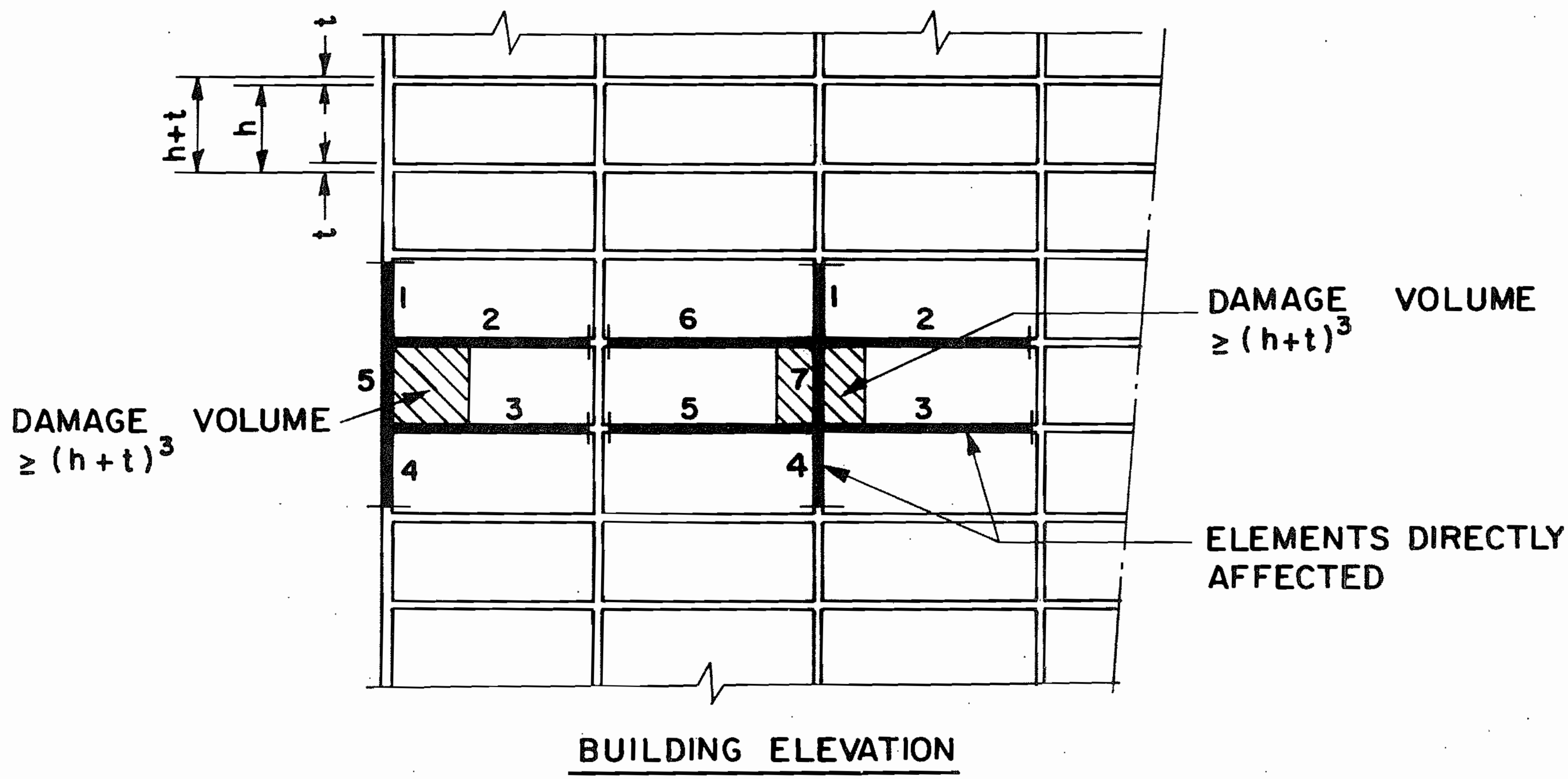

Fig. 3.1 EXAMPLES OF DAMAGE VOLUMES IN A BUILDING WITH TRANSVERSE LOAD BEARING WALLS

Table 3.1 - Minimum Tie Force (SBN 22;35)

No. of Stories	Minimum Tie Force in Pounds/Foot	Connections to be Tied
1 - 4	1400*	All Horizontal Elements**
5	1540	For all Horizontal Elements Whereas all Exterior Vertical Elements must Resist 1400 Pounds/Foot
6	1680	
7	1820	
8	1960	
9	2100	For all Horizontal Elements Whereas All Vertical Elements must Resist a Minimum Tie Force of 1400 Pounds/Foot
10	2240	
11	2380	
12	2520	
13	2660	
14	2800	
15	2940	
16	3080	

* 20 kN/m = 1370 Pounds/Foot, and
2 T/m = 2000 kg/m = 1344 Pounds/Foot.

Therefore, use 1400 pounds/foot as minimum force.

** In addition, the tie force must be greater than specified dead and live load requirements.

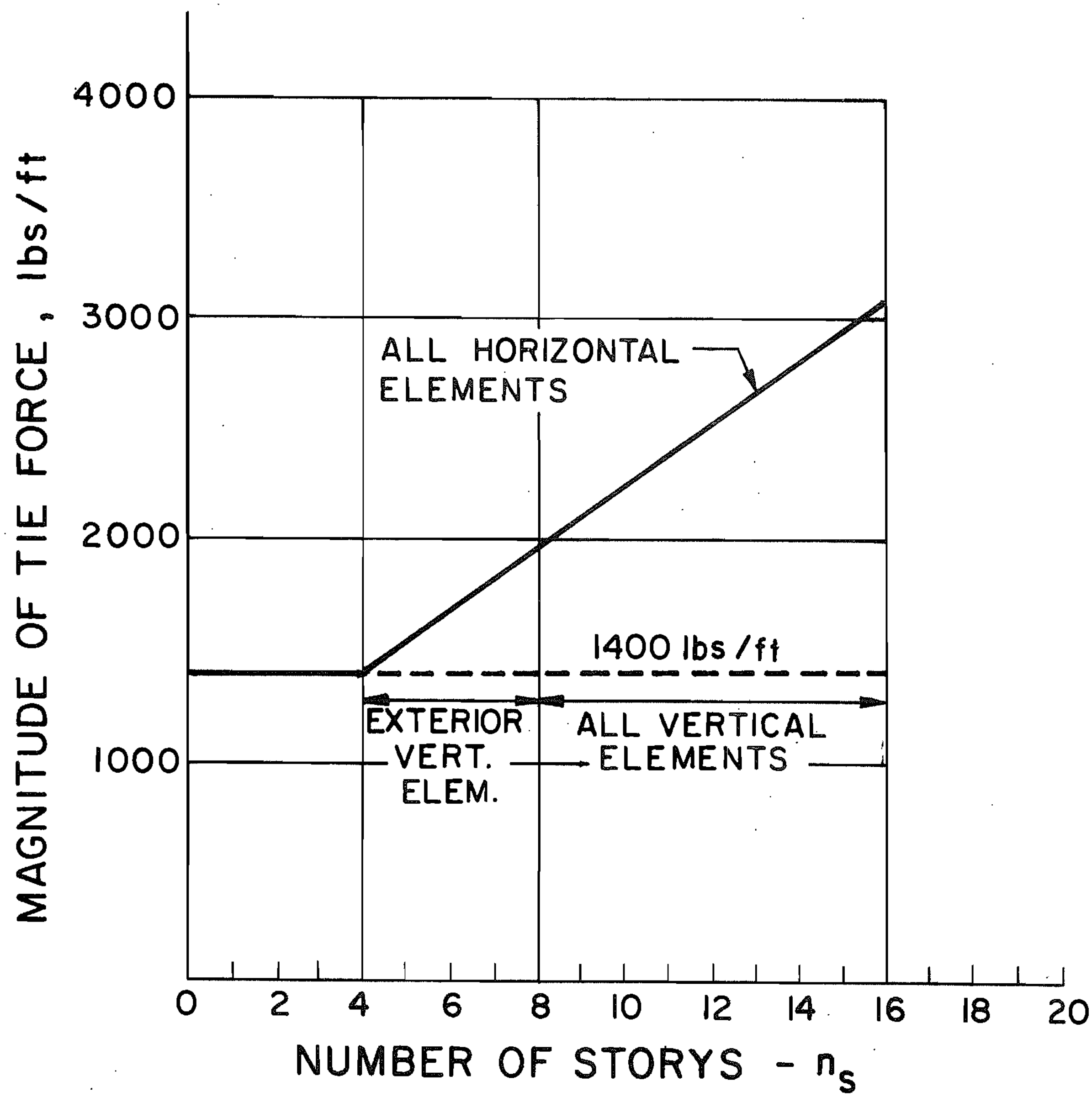

Fig. 3.2 TIE FORCE REQUIREMENTS (SBN 22:35)

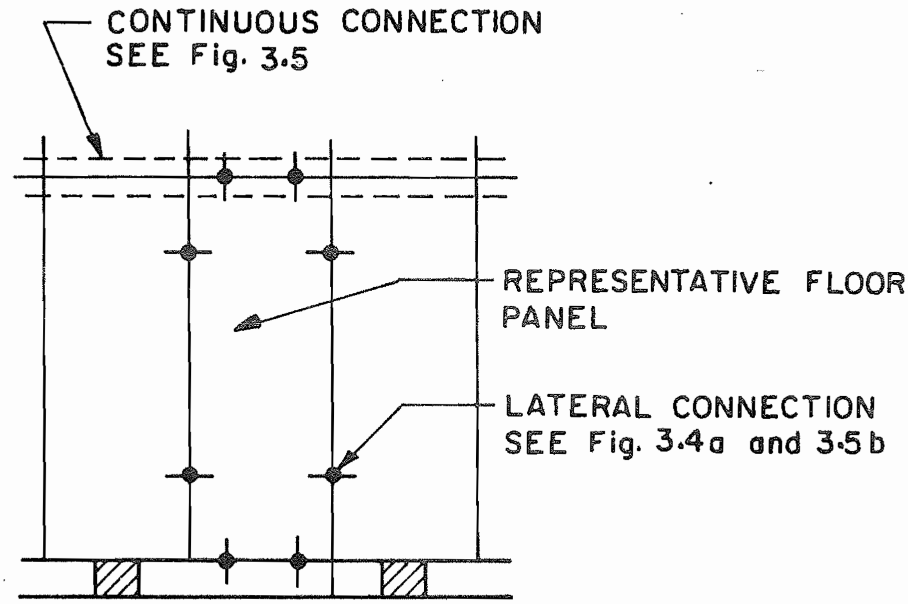

Fig. 3.3 PANELIZED FLOOR LAYOUT

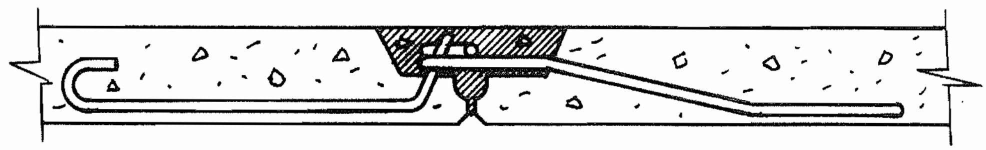

Fig. 3.4a DETAIL OF A LATERAL CONNECTION

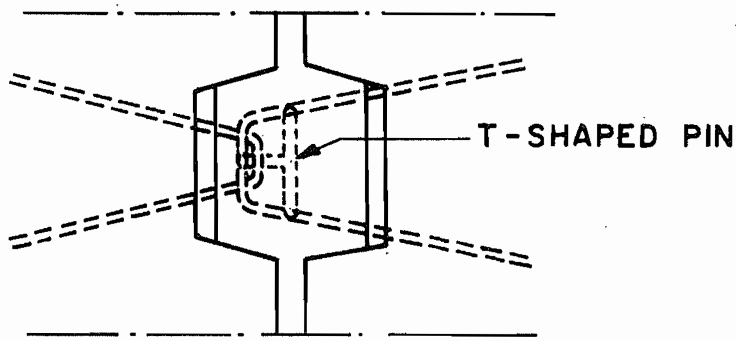

Fig. 3.4b PLAN VIEW OF A LATERAL CONNECTION

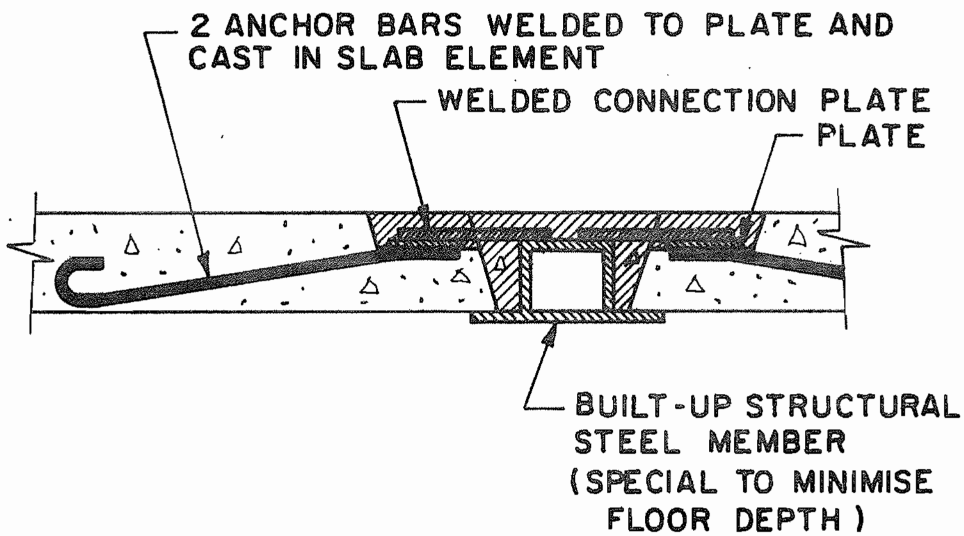

Fig. 3.5 DETAIL OF A FULLY CONTINUOUS FLOOR PANEL SUPPORT JOINT

have serious consequences especially for walk-up buildings with masonry walls and precast concrete floors.

iv) clearly establishes that alternate path and tie force requirements cannot be separated. The provision of tie forces is an essential element-related criterion whereas alternate path considerations involve the overall but damaged structure. This combined requirement is essential as the performance of the damaged structure is very dependent upon the adequacy of the interconnection of those elements not initially damaged by the abnormal loading.

v) introduces the topics of membrane action in floors and cantilever action on the part of the walls of the locally damaged structural system. SBN 22:35 however does not elaborate upon these aspects and apart from the requirement that the ductility of a floor slab should permit an in-plane deformation of two per cent of the span, one has to refer to [3.4] and [3.5] for elaboration. In fact, the minimum tie forces are deemed to satisfy these requirements and in most instances explicit consideration of the mechanics of membrane or catenary action is presumably unnecessary.

Clause 22:3522 is an important clause because it provides a non-specific alternative to the joint application of Clauses :35211 and :35212. Thus the building designer or relevant authority retains some freedom to make decisions. Clause 22:3522 both engenders the development and permits the use of new information and expertise.

3.3.2 Option 2

The intent of Clauses 22:353 and :3531 is unambiguous and reduces to the following choice:

i) either eliminate or reduce the probability of occurrence of an abnormal loading to an acceptable (but unspecified and presumably satisfactory) level, or

ii) design, where necessary, elements and their connections to withstand the abnormal loading or loadings involved. This requires that the loadings be quantified and the behavioral response of the structural elements be amenable to analysis. In SBN 22:35 neither of these aspects are dealt with except that

a) overpressures due to explosion equivalent to a static pressure of from 30 to 40 kN/m^2 are mentioned, and

b) reference is made to the specifications for impact loads on highway bridges.

3.4 Summary Comment with Regard to the Existing Regulations

It is evident that the abnormal loading supplement, SBN 22:35, has had and is still having, a significant impact on the design profession and the building industry in Sweden. While evidence is necessarily subjective the following comments are pertinent:

i) a consensus opinion seemed to be that to comply with these regulations, structural costs would be increased by approximately 7 1/2 per cent but not more than 10 per cent provided the regulations were imposed at the conceptual design stage.

ii) in Sweden, as in much of Europe, there is a trend to use more cast-in-place construction. Whether SBN 22:35 is having an adverse influence on the use of precast concrete is not directly evident, but the whole problem of structural ductility and safety of precast, particularly industrialized, buildings is obviously controversial.

iii) the masonry and light weight concrete (e.g., Siporex, Y-Tong) industries have not yet provided any significant input or response to the regulations.

iv) there is a general awareness of the risk associated with abnormal loading and in particular vehicle impact. Columns in at least two buildings, one of them the new international terminal at Arlanda Airport, were being designed either to reduce the risk or accommodate the impact of a vehicular load.

3.5 SBN 75

Given the nature of the abnormal loading/progressive collapse problem and the inherent conservatism of the building industry it is likely that any regulatory action, particularly if it is enforced, will generate controversy. One inevitable charge will be that the regulations go too far (i.e., in behavior and generality), too fast (i.e., relative to available information and existing design practice). On the other hand advantage can be taken of the inevitable controversy especially if the regulations attempt to be general in scope and reasonably ambitious in intent. For instance:

i) the next or second set of related regulations are likely to be more representative of the industry as a whole and therefore more acceptable and probably more enforceable.

ii) the more "advanced" (i.e., in behavioral and design terms) the next set of related regulations are likely to be, and

iii) the educational aspect of the problem is largely taken care of.

This would appear to be the case with the forthcoming revised Swedish regulations. A draft of the relevant sections of SBN 75 [3.6] has been made available by Dr. Johansson of Statens Planverk but it would be premature to comment on these in any detail. However, the following general aspects of these revised regulations are significant.

i) With the issue of a new code the need for a special supplement to deal with progressive collapse disappears. It becomes easier to define terms, to provide the appropriate context and maintain a consistent perspective.

ii) In SBN 75 the question and definition of abnormal loads* is dealt with in Chapter 7 "Design Loads". Two important features are

a) the explicit and fairly comprehensive treatment of vehicular loading both within the building and at its exterior perimeter. In the event that a precise dynamic analysis is not performed it is required that, where necessary, a specific horizontal force be accommodated. The forces involved vary with the distance between vehicle carriageway

* In the translation prepared by Statens Planverk the phrase Abnormal Loading is specifically and consistently used.

and the element concerned. The forces involved are:

$P = 1500 \sqrt{1-S/25}$ kN	-	for buildings adjacent to a traffic route
$P = 400 \sqrt{1-S/5}$ kN but ≥ 150 kN	-	for cases other than the above but where vehicular traffic can occur
$P = 150 \sqrt{1-S/2}$ kN	-	for vehicular traffic within the building

where S is the distance in metres between the limit of the roadway and the building component under consideration.

b) the following equivalent static gas loadings are specified:

- 50 kN/m^2 (7.5 psi) in rooms without windows,
- 25 kN/m^2 (3.75 psi) in rooms where the window area is 20% of the area of the smallest wall.

Linear interpolation between these values is suggested. The relative magnitude of the load coupled with the recognition of venting is particularly significant.

iii) The regulatory requirements regarding progressive collapse are contained in Chapter 22, Section 33. The intent and approach of this section is essentially the same as that in the supplement SBN 22:35. The main differences between the earlier and revised regulations relate to detail and completeness. For example in the revision

a) the damage volume concept is to some extent de-emphasized and made less explicit and less severe.

b) Instead of the requirements illustrated in Figure 3.1, the simpler criterion of a minimum tie force of 20 kN/m (1400 pounds per foot) in both the horizontal and vertical directions is specified.

To summarize these brief comments concerning SBN 75, it could be said that in comparison with the previous regulations any loss in generality is compensated for by some simplification and additional detail. The intent and even the procedures to be followed are essentially unchanged.

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[3.6] STATENS PLANVERK, An English Translation of extracts from Chapters 21 and 22 of the Draft (1974 - 11 - 20) of the new Swedish Building Regulations, SBN75. (This document was kindly supplied by Dr. Bernt Johansson of the Building Division of the National Board of Urban Planning).

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APPENDIX S: ENGLISH TRANSLATION* OF THE COMMENTARY ON THE SUPPLEMENT CHAPTERS 21 AND 22 IN SBN 67. (SWEDISH BUILDING CODE)

21:93K Exceptional Effects

The requirements of 21:93 with regard to abnormal loadings means that provision must be made for events with very low probabilities of occurrence, namely accidents. In the event of such an incident, a building is not required to withstand intact as for normal loadings and a certain level of damage is acceptable.

The concept of "progressive collapse" (see also 22:3502) is such that the requirements do not apply if a building is so small that the region of primary damage would include the major part of the structure. Single family houses lie within the category of building thus excluded.

"Regard for the risk" implies not only a judgement of the possibility of a progressive collapse occurring but also an assessment of its consequences. The primary intention of the regulations is the protection of human life. Buildings which are entered or approached only occasionally and then only by small numbers of people are thus exempted from the regulations.

An example would be a transformer station closed to the public and visited only for inspection and maintenance. Buildings regularly occupied, even by small numbers of people, should be designed with regard for the risk of progressive collapse, as should buildings which are occupied only occasionally but by large numbers. Examples of the former type are cement factories and boiler houses and examples of the latter are sports stadia.

The requirements of 21:93 are not new in intention but follow from paragraph 42 of the building regulation which states "The building's foundations and structure, and also other components which may be subjected to loading, shall have satisfactory strength, stability and durability".

The formulation of requirements concerning the behavior of damaged structures does however deviate from the general approach of SBN, where the load conditions considered are those that a building should withstand without failure. The basis for measures to satisfy 21:93 should be of a statistical nature and involve assessments of the behaviour

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*Except for minor alterations, this text is essentially Paul Regan's translation [S3B].

of damaged structures. The present regulations are based on currently available data, and further research is necessary to provide date for a better understanding and revision of the present provisions.

:931K Examples of abnormal loadings are given. They include phenomena with differing probabilities of occurrence. Furthermore, the risk associated with any one phenomenon varies greatly from building to building and with time.

Other types of incident can also produce local damage and with it the risk of progressive collapse. Examples are undetected faults in materials, errors in design and construction and unexpected deterioration of materials due to, for example, corrosion or weathering. Such eventualities are however not included in the term abnormal loading in the sense given in 21:93, and damage due to them should be prevented by care in design and construction, by the necessary control at all stages, and by continuing inspection and maintenance. Thus even if the building has been designed with due regard for abnormal loading there is neither reason nor excuse for reduced caution.

a) Explosions can occur in connection with fire, but can also be initiated in other ways. A spark in a switch can be sufficient to ignite an explosive mixture. Explosive mixtures can be formed in various ways. Leakages can occur in gas installations. Liquid gas and volatile liquids such as gasoline are often stored even in ordinary dwellings in quantities sufficient to cause powerful explosions in untoward circumstances.

   Solvents and hardeners for certain types of lacquers and adhesives and mixtures of dust and air have caused severe explosions on a number of occasions.

   The strength of an explosion can vary depending, amongst other things, on the ratios of the components of the explosive mixture, and on its temperature, as well as on the shape and airtightness of the room. It is worth mentioning that the pressure in the well known explosion at Ronan Point, London, has been estimated at between 30 to 40 kN/m^2. In certain circumstances the pressure in a gas explosion can be about ten times this value. See also the report from the Nordic Concrete Society Congress of May, 1970, pp. 159-169.

Section (a) above does not restrict the application of those sections in SBN Chapters 65 and 77 concerned with compartments where the risk of explosion is high and with the handling of explosives.

b) The risk of accidental impact by vehicles exists for many structures. Sections 21:341 and 21:342 take account of those everyday incidents involving a vehicle moving at low speed - e.g., careless reversing, parking maneuvres, snow clearing, etc.

The force produced by vehicle impact can be difficult to determine. It is however evident, with current vehicle weights and speeds, that when the impact is caused by accident, such as one due to the driver's losing control over the vehicle, the force can be greater than those envisaged in Sections 21:341 and 21:342. (See also Section 13.26 of the Specifications for Highways Bridges of the Swedish National Road Administration, which is concerned with damage to intermediate columns in bridges.)

The risk of impact from aircraft is generally very small, but does exist, particularly near air fields. It should be noted that for smaller buildings, the zone of primary damage in the event of such an accident would encompass the whole structure, and consideration of progressive collapse would be meaningless.

The risk of impacts from working machines or from swinging or falling loads is common during construction operations. Especially at civil engineering sites involving heavy transport and large machines, the risk of impact extends to buildings adjacent to the actual sites.

c) Unforeseen settlements are those which in spite of careful soil investigation are larger than or different from those calculated. The term refers primarily to sudden changes in ground conditions, but with some structures slower changes involving differential settlement can significantly affect the load distribution. Plastic deformations and cracking often allow load effects to redistribute over a period of time, but if buildings are inadequately constructed (brittle) sudden failures can occur. When the foundations are on anything other than solid rock, the structure must hav sufficient toughness to eliminate the risk of a progressive collapse due to settlements of unexpected magnitude under parts of the building.

d) Overloading that occurs as a result of carelessness involves a variety of different phenomena, e.g., the application of a live load much greater than that foreseen in design, considerable excesses of maximum loads on lifting gear, and serious damage to the structure caused during alterations.

e) Unforeseen losses of strength in connection with fire can arise for example with the introduction of unexpected flammable materials or unexpected unevenness of fire loading.

22:35 Design for the Prevention of Progressive Collapse

:350K Introduction

The regulations of 22:3502 mean that progressive collapse need be considered only when: (1) the structure has an extent greater than that of the region of primary damage, and (2) there are risks of injury to human beings. The commentary on 21:93 gives guidance on the types of buildings affected by the regulations of 22:35.

The direction of primary bearing for a structural element is defined with regard to behavior of a damaged building Thus a wall may be thought of as acting in a horizontal direction to bridge over a damaged area by cantilever, beam or arch action. Beams and slabs may be assumed to function as membranes spanning primarily in respectively the directions of their spans and shortest spans. The direction of bearing of a column is taken as its length.

:351K General

It should be noted that the regulations of 22:35 are not intended to give complete safety in regard to progressive collapse following any possible exceptional effect in any part of every building. Their intention is limited to a reduction of the general risk of such collapses, as compared to that existing if no such measures were taken.

Alternatively 22:352 can be fulfilled by designing the building in such a way that localized primary damage does not result in progressive collapse. Large deformations can be accepted, and membrane actions in slabs and beams and cantilever actions in walls can be taken into account. Indispensable elements, e.g., some columns, may need to be designed to withstand the abnormal loading in accordance with 22:353.

:352K Measures for Limiting Collapses

The intention of this regulation is that if local damage is produced by an abnormal loading, subsequent failure shall be limited to those parts of the building within and adjacent to the region of primary damage. Other parts of the building are allowed to be severely deformed, but shall hold together so that human life can be saved.

The extent of the primary damage depends on the type and magnitude of the exceptional effect. It is also influenced by the type of structure and the location of the effect within it. In principle, a statistical approach should be employed to judge the reliability of a structure under several different exceptional circumstances. In order to facilitate design the demands of 22:352 can be considered to be fulfilled by compliance with the adhoc requirements of 22:35211 and 22:35212. Such solutions can also serve as guidance as to the performance criteria of 22:3522.

:3521K The recommendations of Section 22:3521 are primarily concerned with current types of multi-story buildings, irrespective of whether or not they are prefabricated. It is intended that as soon as research and development makes it possible, the regulations will be complemented by further practical recommendations. These will deal in a more detailed way with special types of structures, e.g., wall-slab frames, column-beam frames, halls, etc. In this way designers will be given more detailed advice on various building materials and structural systems.

The measures of 22:3521 are intended for use principally in buildings with normal story heights (up to 4 m) such as dwellings, offices, schools, hospitals, etc., and principally in structures with load bearing walls so arranged that local damage can be bridged over by membrane action in the floors and beam or cantilever actions in the walls.

22:3251 is applicable even for other structural systems with well distributed vertical supports and the ability to withstand the horizontal forces arising in membrane actions, although in many cases further consideration of probable behavior will be necessary. For example, the effects of the loss of a corner column cannot normally be controlled by membrane action. The general conditions required for membrane action may be absent in some buildings. The problem can arise particularly in column-beam frames with only two rows of columns in one direction and floors supported on beams in the facades. Such structures and other similar types must therefore be subjected to the special considerations of 22:3522 or 22:353.

Single story buildings with column frames (e.g., halls) can normally be assumed to fulfill the requirements of 22:352 if the columns are encastre at their bases.

:35211K The recommendations of 22:35211 require that the overall stabilizing function should be shared between two or more systems, so that if either of them is damaged, the other can ensure overall stability. If the building already has distributed wind stiffening, for example with several separate stairwells, 22:35211 generally imposes no extra requirements.

If separate overall stability systems cannot be distinguished, the overall stabilizing function must be ensured in some other way, so that local damage to the defined volume/area will not jeopardize overall stability.

For overall stability systems or components with stabilizing functions to be considered separate they must be clearly separated in such a way that they could not reasonably be expected to be damaged simultaneously by the same exceptional effect. As a criterion of clear separation a volume of damage is defined with a cubic shape and a side length equal to the greatest of certain specified dimensions.

In checking total stability after local damage, it is permissible to exploit available safety margins. The allowance of the use of yield stresses or their equivalents indicates the possible extent of such exploitation. For some structures, this indication is clearly not comprehensive and a more detailed analysis of ultimate behavior may be necessary.

In the control of total stability, the wind loading is taken as half the normal design value, and acts simultaneously with other effects including those due to inclinations of "vertical" components as described in 21:91.

:35212K The object of 22:35212 is to stipulate certain minimum requirements regarding the connections (ties) in loadbearing components and in the joints between them, irrespective of the building material and structural system. This creates possibilities of alternative loadbearing actions if parts of a building collapse due to exceptional effects.

These instructions correspond to the general CEB recommendations for "preventing buildings behaving like card houses", which have been adopted by a number of foreign standards with various values or tie forces, and by the Nordic Element Committee. (See the article in "Statens planverk aktuellt Nr. 2/1970".) Relatively small tie forces, of the order of magnitude in question, have been found to give an ability to resist collapse by the development of alternative loadbearing systems - see "Byggnaders stabilitet efter katastrof skador, Modellforsok betraffande krafter i elementfogra". (The stability of buildings after catastrophic damage, Model tests on forces in element joints.) Report No. R20:1971 from Byggforsknings Radat (BFR).

The concurrent requirement of strength in the main loadbearing direction is additionally intended to lessen the risk of damaged parts falling. The loading conditions assumed mean in principle that a structural element loaded with its whole self weight (and for floor slabs with a specified part of the live load) should be able to hang from the reinforcement (or equivalent) providing the joint strength in the main loadbearing direction. This creates the basic requisites for membrane actions, which become effective even at support rotations of 1:5 in slabs with reinforcement in two directions.

For walls the choice of a loaded length equal to twice the story height is an adhoc assumption to allow a reasonable cantilever action in wall panels proportional to the distance between bearing floors.

The requirement of strength in the main loadbearing direction is related to the height of the building, since greater safety vis-a-vis progressive collapse is warranted in high buildings. Even when the minimum requirement (20 kN/m) is decisive for the ties in the main loadbearing direction, the required tie force increases with increasing number of stories.

The reasoning behind the increased tie requirements for higher buildings is as follows:

It is desirable that it should be no more dangerous to live in a high building than in a lower one. If the threshold of damage at which a progressive collapse would commence were as low for a high building as for a lower one, the risks in the former would be greater because the probability of initial damage is greater. Furthermore, the magnitude of possible forces in an exceptional effect generally increases with increased building height. The demand for stronger ties in high buildings is aimed at increasing the load level necessary to produce the primary damage at which a progressive collapse can develop. Thus as far as possible risk should be equalized.

The same intention lies behind the rules given in 22:35211 for distances between local stabilizing systems related to building heights and lengths. By dividing the total stabilizing function between several sub-systems and increasing the level of damage at which overall stability should be maintained as a function of building length, the intention is to ensure that the risk of progressive collapse should not be increased for lengthy buildings.

These rules are now based on certain general judgements and reasoned assumptions. The requirements should be adjusted in the light of statistical data, as and when these become available.

"Parts of the building frame, whose failure, separation or removal could cause progressive collapse" include balconies, stairflights, etc. In such cases care should be taken to ensure that the collapse of such a component cannot start a continuous collapse of similar elements below it (balconies below one another, etc.)

Ductility of connections at joints is necessary for the ties to function as intended. This point is commented on in the NEU statement, which includes a note that "the following example can give some indication of the possibility of membrane action and the requirements of deformations at joints".

A slab, which has its flexural stiffness greatly reduced by damage, can still carry its load by membrane action if certain conditions are fulfilled. The average deformation in the plane of the slab at the level of the reinforcement is required to be of the order of two per cent. A considerable part of this deformation must occur relatively locally at the supports, unless the slab has specially favorable material characteristics.

This places considerable demands on the ductility of ties at joints, in cases where deformations must be accommodated in short lengths, so that only materials with high ultimate strains can be used. The good bond of deformed bars can be a disadvantage so far as strain distribution is concerned, especially in dynamic loading conditions which can arise in the present context. Unless special measures are taken to destroy bond or supporting test data can be cited, it is suitable to use plain bars (or their equivalents) as ties at joints.

The requirement for ties in the vertical direction applies only to buildings with more than four stories (basements largely above ground included in accordance with the definition of a story in paragraph 37 of the building law), and the requirement of strength (corresponding to a minimum) is dependent on the number of stories above four. Up to and including eight stories the requirements apply only to components in the facade, external loadbearing walls, external columns, edge beams, etc. The reason for these limited demands applying only to the exterior walls is partly that these are generally the most significant in regard to stability after damage, and partly that they are commonly the most liable to suffer a number of types of exceptional effects.

:3522K This section concerns cases where 22:352 is applicable, but a strict application of 22:35211 or 22:35212 is not possible. In such cases it must be shown that for the chosen type of building the stability after damage is satisfactory in regard to the general requirements of 22:352. In determining this, the cubic volume of damage defined in 22:3521 can be used as a guide to the connections (ties) needed between "components of the building frame".

In principle it is to be assumed that all building components within the volume of damage lose their loadbearing capacities. Under certain circumstances however, indispensable components can be designed to resist exceptional effects (see 22:353). Also, to some extent, measures reducing the probability of exceptional effects can be accepted. It is however not intended that the use of this section should involve an analysis of the total situation with regard to risks and preventive measures. The "approval of the relevant authorities" can, in some cases, be replaced by a general approval of a type of construction shown to be reliable.

:353K Measures for Reducing the Risk of Damage Due to Exceptional Effects

The measures of 22:353 are intended, for example, for buildings or parts thereof with exceptionally strong frames or for special purpose buildings or such a configuration that collapse subsequent to local damage cannot be limited by the measures of 22:352. Examples are towers, buildings without conventional loadbearing systems such as cantilever and suspended structures, some column frames and industrial buildings used for the handling of explosive substances.

The rules of Chapters 65 and 77 SBN are to be applied to the design of areas where there are special risks of explosions. That a building is designed with regard to the risk of progressive collapse does not affect their applicability.

:3531K The vulnerability of a building is to be judged with regard to the actual risk of exceptional effects in accordance with 21:93 and when possible in the light of information which future research work may give on the order of magnitude of conceivable consequences. As a general principle, all the components that are required to remain intact, if a serious collapse is to be avoided, are to be made of sufficient strength to render negligible the risk of failure due to exceptional effects. It should be noted that columns are relatively insensitive to explosions but need considerable stiffness, strength and mass to resist impact. In structures where loads are supported by tension special consideration should be given to the possibility of reducing the risk of collapse due to local damage by distributing loading between several tension members.

It is intended that 22:3531 should be applied only after consultation with the relevant authorities during which the assessment of vulnerability and/or risks of exceptional effects should be investigated.

CHAPTER 4: DENMARK

4.1 Background and Pertinent Regulations

In September, 1968*, a committee was set up by the Danish Association of Engineers to investigate the safety of buildings against local overloading. At the request of the Ministry of Housing this Committee developed a number of design requirements and these were published as Supplement No. 3 (dated February, 1969) to the then existing Building Regulations [4.1]. The Building Regulations have since been revised but in the most recent version, dated June, 1972, the clauses pertaining to abnormal loading remain essentially unchanged. A translation of these clauses follows:

BYGNINGSREGLEMENT (Building Regulations)
BOLIGMINISTEREIT (Ministry of Housing)

5.2 Design (Dimensioning) of Building Structures

Clause 1. Building Structures shall be designed on the basis of the Codes for Building Structures of the Danish Association of Engineers.

a) Load Specification, DS140
b) Concrete and Reinforced Concrete Construction, DS411
c) Steel Construction (DIF Code No. 15)
d) Timber Construction, DS413
e) Masonry, DS414
f) Foundations, DS415
g) Hollow Core Floor, DS416
h) Horizontal bearing elements of reinforced lightweight autoclaved foam concrete, DS420, 1
i) Load Bearing Panels of "Wood-Concrete" Construction, DS422
j) Concrete Hollow Block (DIF Code No. 67)

Clause 2. Buildings of more than six stories shall be designed and executed in such a manner that either requirement (a) or requirement (b) below is satisfied. Usable roof areas and cellars, whose ceilings lie more than 1.25 metres above ground level, are to be counted as stories:

a) in any room bounded by an exterior wall, the floor or the roof and/or the exterior wall shall be able to fail completely without leading to the failure/collapse of any

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* The building system used at Ronan Point was essentially of Danish origin, i.e., Larsen and Nielsen.

other story divider (floor or roof) other than those bounding the room in question. Where the floor area is divided by non-load bearing interior walls perpendicular to the span of the floor, the area between floor supports shall be considered as a single room.

b) *every normal cross-section in the load bearing exterior wall or in the story divider (floor or roof) shall be designed to resist a tensile force of 20 kN/m (2Mp/m) without exceeding ordinarily permissible stresses. The mitigating or beneficial effects of compressive loads normal to the section shall be disregarded but, in sections subject to tension, the combined tensile effects must not exceed the ordinarily permissible level of stress. The necessary reinforcement may be partially comprised of the reinforcement within the element or in the joint but the reinforcement in the joint must be deformed bars in accordance with DS411.*

Splices and anchorage lengths shall be in accordance with DS411. In the event that at any cross-section more than 50 per cent of the reinforcement is lapped, the lap or anchorage lengths required by DS411 shall be increased by 50 per cent.

4.2 Design Procedures

Without specifying cause these regulations, in order to avoid overall structural failure due to a localized loading, prescribe either the use of the alternative path approach or the provision of tie reinforcement to resist a tensile force of approximately 1400 pounds per foot width. Because of the lack of design guidelines, the novelty of the alternative path approach, the relatively low value of the tie force and the self evident desirability of structural continuity, the first option is rarely chosen.

In physical terms the 1400 pounds per foot tie force is approximately equivalent to one #2 reinforcing bar at 2'0" intervals or a #6 bar every 10 feet of joint or wall or floor slab. It is estimated that the economic impact of these regulations has been slight - of the order of less than one per cent and certainly less than two per cent of structural costs. While the quantitative requirements of these regulations are by no means onerous the specification of minimum tie forces and thus, a datum for structural continuity, has had the effect of compelling the designer to consider in some detail the structural performance of exterior walls and all floor joints. The joint illustrated in Figure 4.1 is one example of a joint providing vertical and horizontal continuity. Two related and beneficial developments are:

i) the apparent tendency for buildings with less than six stories also to be designed for minimum tie forces, and

ii) the probability that when Danish building systems are exported particularly to those countries without safety requirements specifically for abnormal loading, Danish practice will prevail.

While the alternative path option may be chosen only rarely to design a building, the very fact that this approach is mentioned and permitted is of considerable consequence. For example, the effects of localized loading are enunciated, the vulnerability of exterior walls is emphasized, the concept of avoiding progressive collapse is stated and the limits of acceptable damage are to some extent quantified. Clause 2a provides an incentive to investigate failure theories and undertake research. Probably the most important influence is on the layout of the load bearing structural system. In Figure 4.2 is shown the layout of a representative apartment building while in Figure 4.3 are shown some of alterations that may occur as a consequence of considering the alternative path approach. For example:

- to reduce the vulnerability of the flank wall and limit the extent of actual or stipulated damage some form of cross-wall perpendicular to and integral with the flank wall may be provided;
- returns on load bearing walls are advantageous when considering the behavior of the system minus either a wall or a floor element;
- the use of transverse walls or strategically placed beams is also advantageous in avoiding progressive failure as a consequence of a localized incident.

One possible criticism of Clause 2a is that the extent of damage i.e., the portion to be considered removed, is imprecisely defined. For example, in Figure 4.2, the damaged floor area of the "room" could be the whole floor area between the flank and the first interior row of transverse

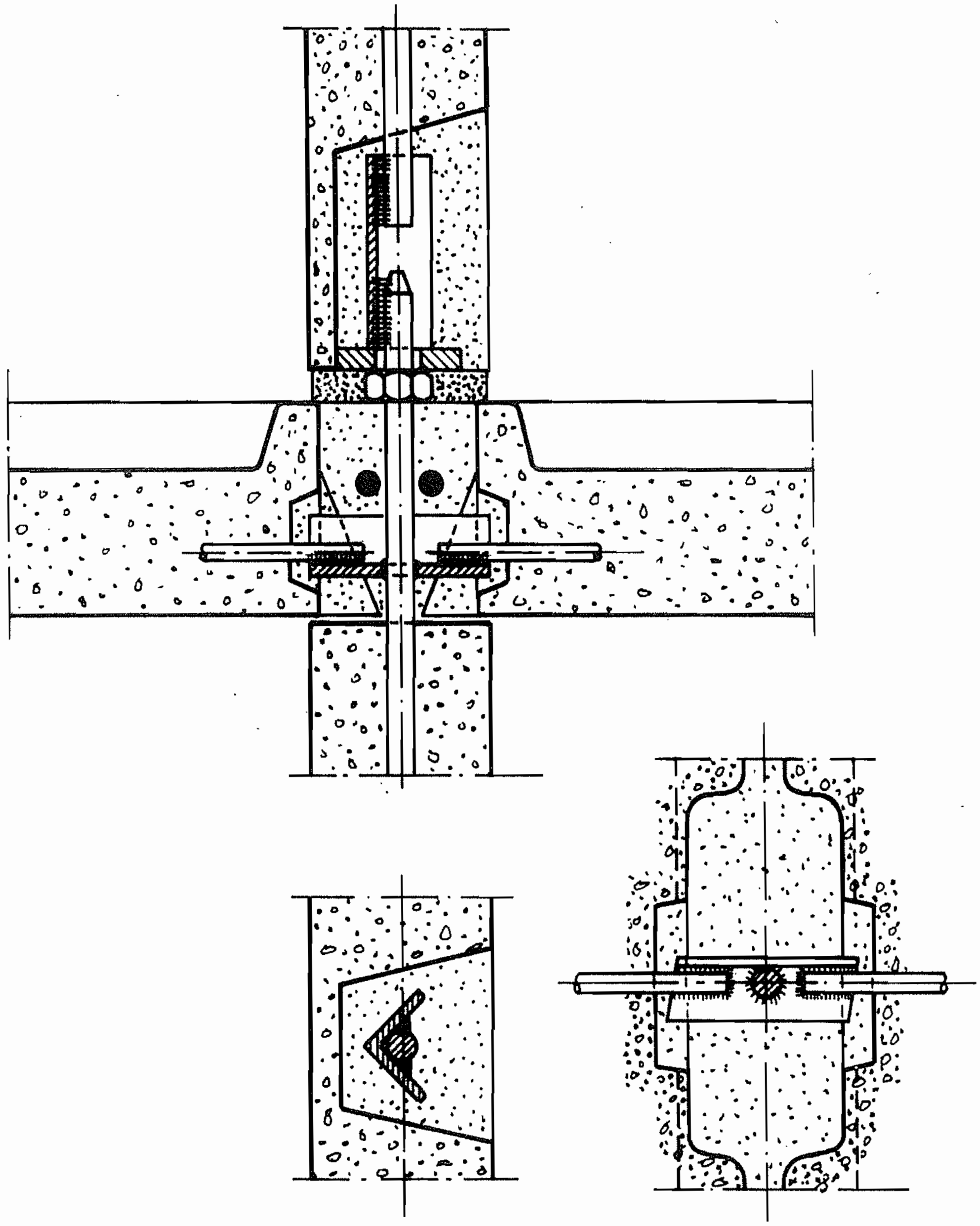

Fig. 4.1 - REPRESENTATIVE JOINT DETAIL [D3]

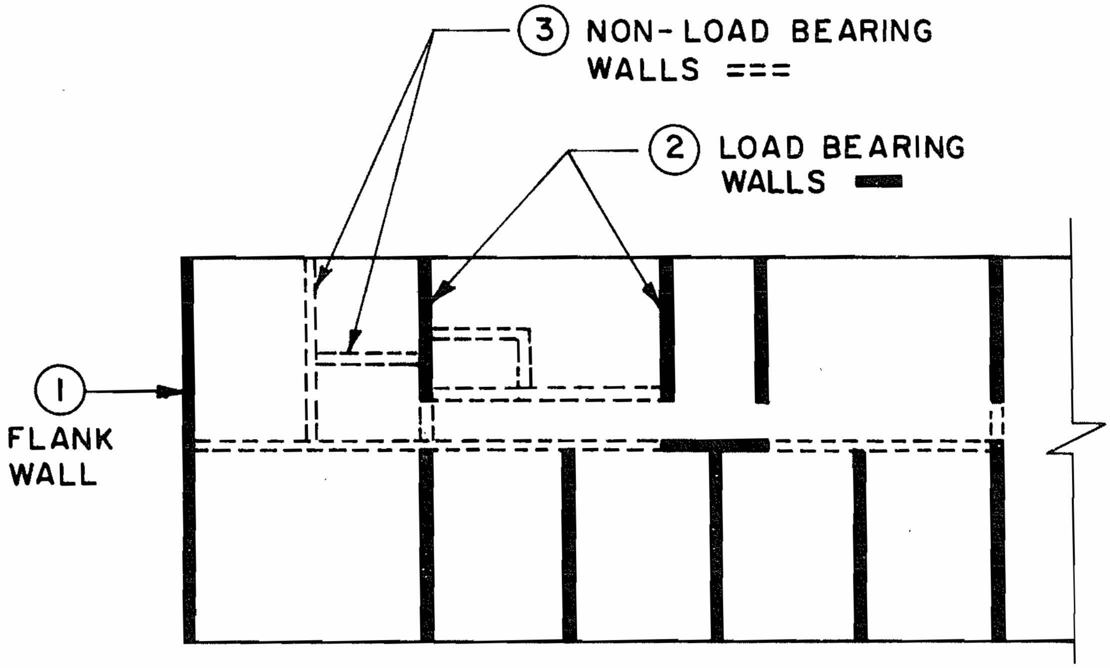

Fig. 4.2 REPRESENTATIVE APARTMENT BUILDING LAYOUT.

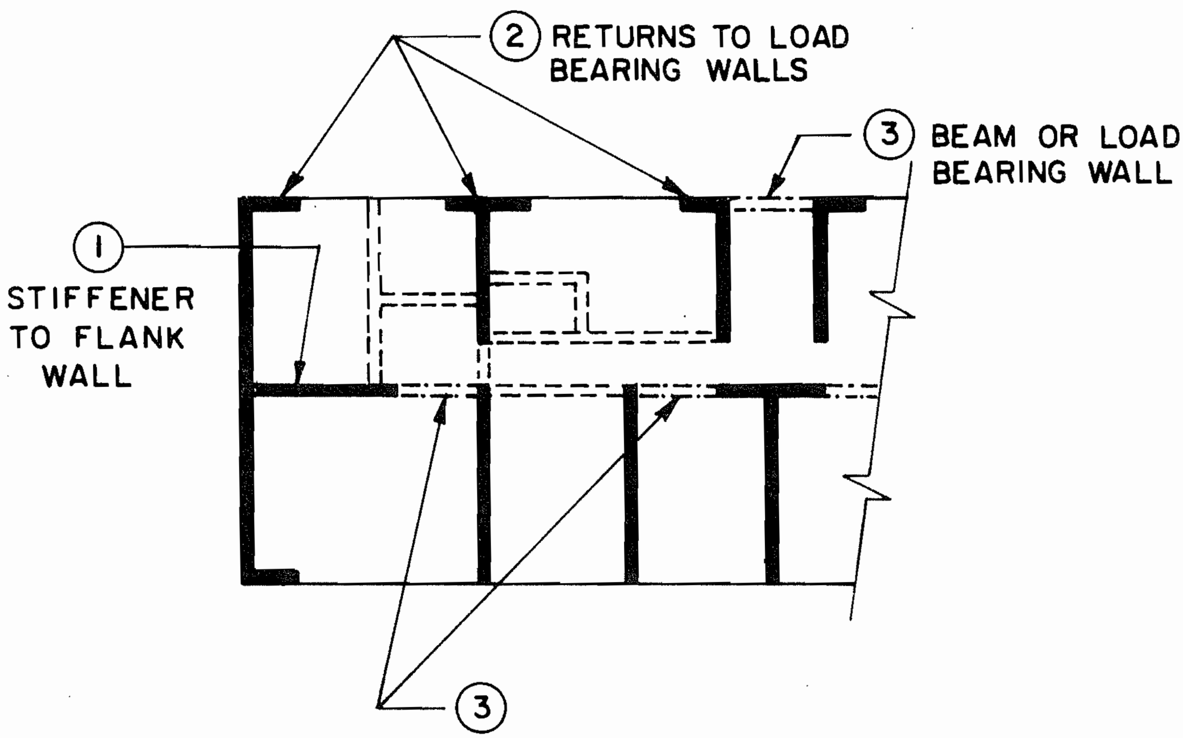

Fig. 4.3: POSSIBLE IMPROVEMENTS IN LAYOUT FOR PURPOSES OF STRUCTURAL SAFETY.

walls. While only those "rooms" bounded by an exterior wall need to be considered it is possible for the local failure of an interior load bearing member to have serious consequences and in a Committee Report dated May 1, 1969 [4.2] it was suggested that the load bearing walls adjacent to stairwells also be treated as exterior walls.

Neither stress nor load levels are specified when considering the alternate path approach but in the Committee Report [4.2] it was stated that after local overloading the building must avoid overall failure long enough to evacuate the inhabitants. Evidently, life safety is the principal requirement and by implication somewhere between two hours (or the fire rating) and 48 hours (to permit inspection and propping) is the length of time the building should remain intact. Accordingly, the Committee suggested that an 80 per cent increase in permissible (i.e., normal service load) stresses be permitted and only one third of the gravity live load and one quarter of the service wind load be used for post-local failure considerations.

These regulations apply essentially to bearing wall systems over six stories in height. Because very few masonry buildings of this height are built and because cast-in-place concrete will, in most instances, automatically comply with Clause 2b of these regulations, in practice only precast systems will be significantly affected by these clauses.

4.3 Summary

The Danish regulations are of particular interest in that, with a modicum of words,

i) the consequences and, thus, the existence of abnormal loadings, (presumably explosion and impact) are acknowledged but issues such as their magnitude, nature, incidence and associated risk are completely avoided.

ii) the desirability of effective continuity and the tying together of structural elements is emphasized and pragmatically assured by the specification of a nominal tie force criterion.

iii) the larger and much more complex issue of the performance of the damaged structure is brought to the attention of the designer. Having permitted the alternative path approach the impetus to consider the overall stability problem, to undertake more detailed design and to do additional research and development is maintained.

These regulations could be criticized on a number of grounds. However, they do constitute a viable interim specification. They are neither an overly complex nor an unduly simplistic attempt to cope with the structural implications of abnormal loadings, progressive collapse and building safety.

CHAPTER 4: REFERENCES

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[4.3] "Problems of Overloading", Larsen and Nielsen News, No. 52, November, 1973, pp. 12-13.

CHAPTER 5: WEST GERMANY

5.1 Introduction

The principal code of practice for the design and construction of reinforced concrete buildings is DIN 1045, "Beton und Stahlbetonbau; Bemessung und Ausführung" [5.1]. A revised version of this "Deutsche Normen" was issued in January, 1972.

In Section 15.8.1 which is concerned with the general requirements for spatial stiffness and stability, it is stated that:

> *"Special consideration must be given to the spatial stiffness and stability of buildings. Structures in which the failure of one element can lead to the progressive collapse of other elements, are to be avoided if possible. If the stiffness and stability of a structure is not immediately obvious, calculations must be made to ensure the ability of the horizontal and vertical stiffening elements to 'stand up'; with due regard for tolerances and vertical load eccentricities in accordance with clause 15.8.2."*

The remainder of Section 15.8 is concerned with more familiar aspects of stability but it is significant that, without specifying cause, the structural phenomenon of progressive collapse is recognized and its avoidance recommended. This statement appears to be the only explicit reference to progressive collapse in any of the pertinent regulations. On the other hand there are numerous statements and requirements that implicitly have a direct bearing on the ability of buildings to accommodate abnormal loadings and avoid progressive collapse. As a number of codes or norms are involved it is advantageous to consider loadings first, i.e., cause, and then to consider the structural requirements to accommodate their effect.

5.2 Loading

Section 19 of DIN 1045 applies specifically to buildings utilizing precast elements and in Section 19.1 the following is stated:

"Design in accordance with clause 15.8.1 is especially important in buildings composed of precast elements. Load carrying and stiffening precast elements are to be connected to each other and to cast-in-place elements by reinforcement or other equivalent means such that extreme loading (settlement, severe vibration, fire loading, etc.) does not cause instability."

Thus the progressive collapse aspect of stability is emphasized. While the existence of an extreme loading is recognized the example loadings listed are, in terms of the loading classification system of references [1.1], [1.4], extreme values of normal forms of loading. Neither explosion nor vehicular impact are mentioned although, of course, it could be presumed that the "etc." also incorporates all abnormal forms of loading.

As far as buildings or building loads are concerned there is apparently no explicit mention of service system·or bomb explosion loading in any of the relevant codes. However, vehicular impact is dealt with in considerable detail. For this reason, the regulatory treatment of vehicle collision will be considered separately.

5.3 Vehicular Impact

DIN 1055 "Lastannahmen, Blatt 3 - Verkehrslasten" (Load specification, Section 3 - Live Loads) [5.2] issued in June, 1971, is particularly relevant. Usually the design load is specified as an equivalent static force acting horizontally. A variety of situations are covered in Section 7 and the following, freely translated, clauses are illustrative of the scope and nature of this Section of DIN 1055.

7.4.1 Horizontal Impact on Load Carrying Columns and Walls

7.4.1.1 Near Streets and Roads

In built up areas those buildings within 1m (3'-3") of the curb are to be designed to accommodate horizontal impact at a height of 1.2m (4'-0") above ground level acting once in the direction of travel and, separately, once perpendicular to the direction of travel. The equivalent static force involved is 500 kN (112 K) at all projecting corners and 250 kN (56K) on all other vulnerable load bearing elements. If it can be shown that the failure of the load bearing element under consideration does not impair the

> *stability of the overall structure then it is not necessary for the element to be designed to take the above impact loads.*

Similar requirements are specified for service stations, parking garages, warehouses, churches, stadiums, etc. Even scaffolding, non-loadbearing elements and parapets are covered. In addition, sub-section 7.4.3 specifies permissible stress levels for reinforced concrete, structural steel and masonry buildings.

Of comparable interest is the addendum dated January, 1972, to DIN 1072 - "Strassen - und Wegbrücken" (Road and Highway Bridges) [5.3] where Section 7.2 requires that:

> Loadbearing columns, framing members, end members of trusses, etc., are in general to be designed for vehicular impact or provision must be made to avoid impact. For roads in built-up areas (maximum speed 80 km/hour) the impact loading, acting in concert with the most unfavorable combination of the other loads, occurs at a level 1.2m (4'.0") above the pavement and should be taken as:
>
> ± 1000 kN (225 K) in the direction of travel, and/or
>
> 500 kN (112 K) perpendicular to the direction of travel.

As in Section 7.4.1.1 of DIN 1055 an exception is made if the reinforced columns or walls are sufficiently massive to accommodate some impact damage without impairing the overall stability of the bridge.

Between the various West German codes of practice the design problems associated with vehicle collision are treated in considerable detail. The prevalence of this particular abnormal loading is recognized, equivalent loadings and performance requirements are specified and by providing upper bound stress levels the nature of behavioral response is tied in with normal design procedures. Moreover, by implication, for vulnerable columns in both buildings and bridges the alternative path approach is effectively specified as an alternative to designing for a specified impact load.

Two points worth emphasizing are:

i) the magnitude of the collision loads to be resisted (between 225 and 56 K) is large;

ii) whether buildings or bridges are involved the West German specifications are much more specific, detailed and onerous than existing U.S. regulations for vehicular collision.

5.4 Tie Requirements (DIN 1045)

The collision of vehicles with buildings is dealt with in DIN 1055. DIN 1045 does not differentiate between the various forms of abnormal or the extreme forms of normal loading. Because specific loadings are not identified, these loads must be accommodated indirectly and in a general and collective manner. DIN 1045 attempts to satisfy clause 19.1 by prescribing minimum tie forces and limiting the use of precast elements. The intent of the relevant clauses will be discussed with reference to wall and then floor elements. For a more detailed discussion of the relevant sections of DIN 1045 reference should be made to a paper by Manleitner [5.4].

5.4.1 Precast Walls

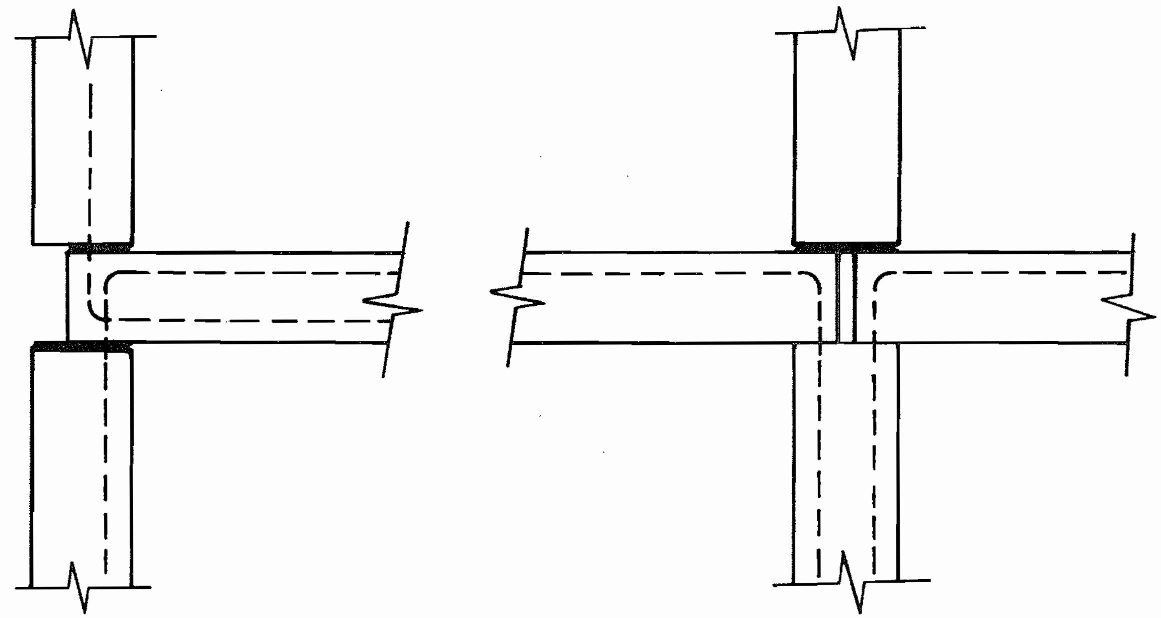
Section 19.8.1 makes the following general provisions:

- the regulations pertaining to cast-in-place construction also apply to precast construction unless otherwise stated.
- structural elements must be made up of story height elements only, except in the neighborhood of stairways.
- if the precast elements are required to withstand vertical loading, horizontal loading or both, then the ability of the joints to resist and transfer the forces involved must be demonstrated by calculation.
- precast wall elements of width less than the story height and narrower than the spacing of stiffening walls, may be used to comprise shear walls in buildings of more than three stories only if adequate proof is provided.

A high-rise building is defined as one in which the floor of at least one room is more than 22m (72 feet) above ground level. Section 19.8.6 then specifies that in high-rise buildings all exterior structural wall elements must be connected at both their upper and lower edges to the adjoining floor slab via reinforcement or other steel means. These connections are to be designed for a tensile force of at least 700 kp/m (470 pounds/foot) of wall acting perpendicular to the plane of the wall. Interior structural wall elements need only be connected at their upper edge by means of reinforcement with a cross-sectional area of at least 0.7 cm^2/m (0.033 ins^2/foot). In non-high rise building only the exterior structural wall elements need be connected at their upper edges to adjoining floor slabs. Tie reinforcement is to be fully anchored and the allowable stresses are not to be exceeded. The horizontal distance between connections must not exceed 2m (6'-7") and any connection should not be more than 1m (3'-3") from a vertical edge.

The prescribed force of less than 500 pounds per linear foot is not quantitatively onerous and leads to much less reinforcement than the British CP 110 for example. Although it is not stated, the tie force requirement is presumably additional to any other function that the joint reinforcement may be required to provide. Perhaps more emphasis could be placed on the need for continuity of the tie within the elements as opposed to that between elements.

DIN 1045 requires that wall elements be connected to floor elements but there is no explicit provision for tie reinforcement in the vertical plane between vertical elements. This could be a serious deficiency. For instance, Section 19.8.6 calls for wall to floor continuity only and it would be possible merely to provide the tie reinforcement illustrated in Figure 5.1. Apart from the inadequacies of joints of this type in accommodating explosive loading (see Rhodes [2.23], the possibility of obtaining joint details such as Figure 5.1.B is potentially dangerous. Particularly so for uplift due to an explosion, or for that matter, seismic action. Even more serious is the fact that if the lower wall element were somehow to be removed or damaged, the floor elements above would collapse since they are not tied to the upper wall element and extensive damage could result. In order for bridging or catenary action to develop in the damaged structure (alternative path approach)



A) EXTERIOR WALL TO FLOOR CONNECTION.

B) INTERIOR WALL TO FLOOR (INFILL JOINT) CONNECTION.

Fig. 5.1 TIE PROVISIONS - SECTION 19.8.6. DIN 1045 it is essential that vertical continuity be ensured. Although this is not explicitly required in DIN 1045 at least two major panelized building systems in West Germany, Compta and Dressler Spannbeton BmbH, provide wall to wall as well as the required wall to floor continuity.

Section 19.8.3 deals with the possible interconnection of wall elements that are perpendicular to each other. If for reasons of stability the continuity provided along the upper and lower edge of the wall element is inadequate and it is necessary to tie this element to a transverse wall element, then it is recommended that:

i) the tie reinforcement be provided at third points within the vertical joint,

ii) each of these ties should be capable of resisting a tension force of one per cent (1%) of the vertical loading taken by the wall element involved.

It is obviously good practice to tie all vertical structural precast elements together. However as no specific extreme normal loading or abnormal loading is mentioned or quantified it is difficult to evaluate stability precisely. Thus it is probable that designers comply merely with specified tie reinforcement requirements. Explosive loading could not have been uppermost in the minds of the code writing body when compiling Section 19 of DIN 1045. In this qualitative sense these regulations have some shortcomings.

5.4.2 Precast Floor and Roof Slabs

Section 19.7.1 prescribes in considerable detail where precast horizontal elements may or may not be used. The principal intention appears to be to limit the use of panelized floors and roofs to those buildings with essentially static and uniformly distributed loads or low magnitude dynamic or non-distributed (e.g., passenger car) loading. In addition to a suggestion that floor panels be as wide as the room involved, in large panel buildings it is required that floor panels be not less than 2m (6'-7") in width. This would appear [5.4, 5.5] to be an indirect measure to avoid progressive collapse. In general room sized floors would span in two directions and in the event of the failure of any support the probability of the floor collapsing is much less than would be the case with narrower panels.

Sections 19.7.4.1 and 19.7.4.2 are concerned with the interconnection of floor elements and the need for the floor system to act as a diaphragm. For example, section 19.7.4.2 requires that, in addition to any other reinforcement, reinforcement be provided within the joint between floor elements over all interior structural walls. This tie force must be able to sustain a tensile force of at least 1.5 Mp or 15 kN (3300 pounds). In the event that the floor element is less than room size, reinforcement must also be provided in the joints (presumably within the infill joint parallel to the main span). This reinforcement must resist a total tension force of 15 kN (3.3K) and must be connected to other reinforcement in the floor slab. Thus the continuity of the tie within and between floor elements is preserved. This aspect is emphasized by a statement in 19.7.4.2 to the effect that this continuity must be maintained at corners, recesses or holes, etc.

Section 19.7.4.1 attempts to ensure that a 'ring anchor' or continuous tie is provided around all floor areas of not greater than 150 m^2, i.e., not more than 10m, (33 feet) on the shorter side and a longer side of not more than 1 1/2 times the length of the shorter side. The tie force to be resisted is 3Mp or 30kN (6600 pounds) and it is further suggested that at least two 12 mm (2 #4) reinforcing bars would be satisfactory. Figures 5.2 and 5.3 illustrate the requirements of these sections of DIN 1045.

It is evident that the tie requirements for the floor system are reasonably comprehensive but not particularly onerous in terms of quantity. One point to be noted is that all the tie reinforcement can be placed in the joints between floor elements, i.e., in the "natural" joint. This is coupled with the requirement that these floor elements be wider than 2m (> 6".0") so that joints parallel to the span and hence the tie reinforcement will be at least this far apart.

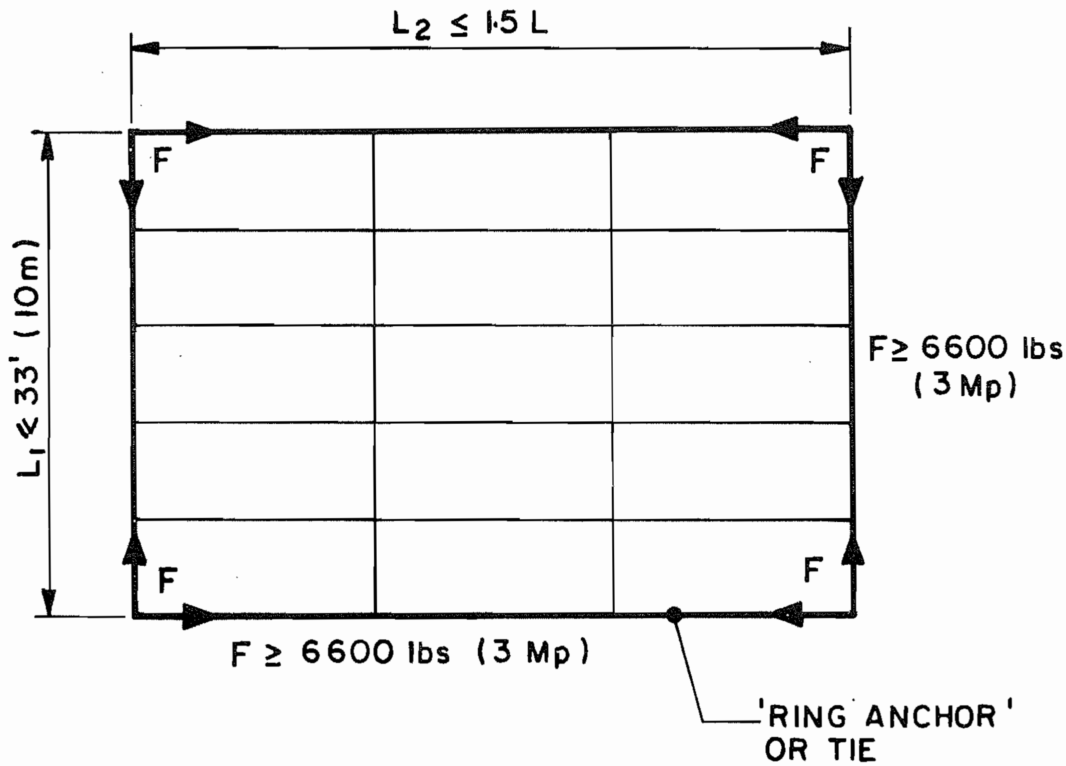


Fig. 5.2 PROVISIONS TO ENSURE DIAPHRAGM ACTION IN PANELIZED FLOOR SYSTEMS [5.4]

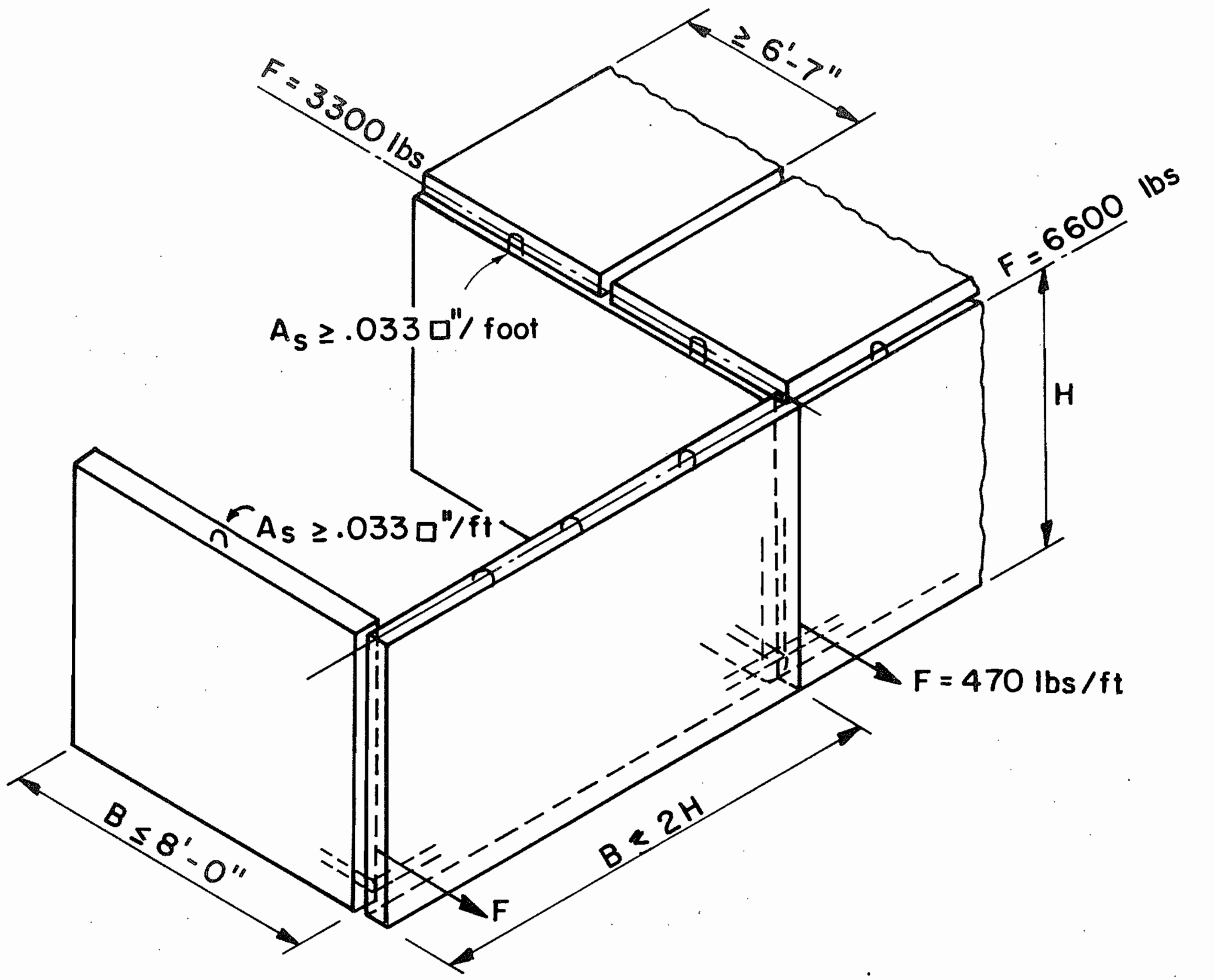


Fig. 5.3 CONNECTION OF WALL PANELS TO FLOOR DIAPHRAGMS ACCORDING TO DIN 1045 AND DIAPHRAGM REINFORCEMENT ACCORDING TO SECTION 19.7.4.2 [5.4]

5.5 Summary

Taken together, the relevant sections of various West German codes of practice do recognize the significance of extreme normal loads, if not abnormal loadings, and suggest that progressive collapse should be avoided.

The only abnormal load that is identified and treated fairly comprehensively is vehicular collision. Probably DIN 1055 is the most detailed of all Western European codes in this respect. The impact forces are specified as static equivalents, permissible stress levels are stated and, by implication, even the alternative path method and the performance of the damaged structure are prescribed as an alternative design approach. This aspect of the German regulations deserves more detailed study.

As far as extreme and/or abnormal loadings in precast panelized buildings are concerned, DIN 1045 treats the problem in an indirect but explicit manner by dealing collectively with all loadings and requiring tie reinforcement to resist specific tensile forces. The fact that the regulations are imprecise with regard to the vertical tie between wall elements and that explosive loading does not appear to have been considered, may detract from the adequacy of these regulations.

CHAPTER 5: REFERENCES

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[5.2] "Lastennahmen, Blatt 3 - Verkehrslasten", Deutsche Normen, DIN 1055 Blatt 3, Deutscher Normenausschuss, Berlin 30, June, 1971.

[5.3] Ergänzungsbestimmungen von January 1972 mit Ergänzerdin Erlauterungen zu DIN 1072, "Strassen - und Wegbrucken", Ausgabe, November, 1967.

[5.4] MANLEITNER, S. "Richtlinien und Bestimmungen der neuen DIN 1045 für Grosstafelbauten" (Directives and specifications for the new German Standard DIN 1045 for large-sized slab constructions), Betonwerk & Fertigteil-Technik, Heft 1, 1973, pp. 5-14.

[5.5] Private Communication from Prof. Dr. Ing. Gerhard Mehlhorn and Dipl. -Ing. Heinz Schwing, (April 28, 1975).

CHAPTER 6: NETHERLANDS

6.1 Background and Pertinent Regulations

Under the section entitled "General Considerations and Loading" in the Dutch Building Regulations, NEN 3850, explicit mention is made of the possibility of explosion and vehicle impact and the attendant need to avoid catastrophic failure. A translation of the pertinent clauses and related commentary is as follows:

NEN 3850 "Technische grondslagen voor de berekening van bouwconstructies - TGB 1972" (Regulations for the design of building structures).

3. *Special Influences*

   a. *The loadbearing structure should be constructed in such a way that localized damage cannot have catastrophic consequences. Local damage to a structure can be caused by fire, explosion, vehicle collision, vibration, etc.*

   b. *In some of these cases the distribution of forces in the loadbearing structure may be also influenced by temperature differences, shrinkage, creep, freezing, unequal settlements, etc.*

Commentary

3. Structural damage is frequently caused by an accident such as fire, explosion and collision; also excessive loading or faults in the materials can result in local damage (see F.K. Ligtenberg: "Veiligheid en Catastrofen", TNO-Nieuws 1969, No. 3) [6.1]. Damage of limited extent can in some circumstances lead to great havoc or cause extensive property damage. In many cases this can be foreseen and may be avoided by relatively simple structural measures. There are instances where the magnitude of the forces that are exerted on the bearing structure are dependent upon the strength of secondary structural elements that will, in any event, fail (e.g., in the event of an explosion or in the case of imposed deformations). In buildings of some importance the loadbearing structure should be so strong that failure of a secondary structural element (such as an infill wall or roof deck) does not result in disproportionately extensive damage.

While the need to avoid catastrophic failure due to abnormal loading is explicitly stated, no guidance is provided nor are the abnormal loading types, explosion or vehicular impact, quantified. Clause 3.a furthermore implies that there should be zero probability of catastrophic failure. It is evident that while the building regulation does identify the problem, its resolution is left largely to the discretion of the structural engineer. Fortunately, a considerable amount of work on building safety has been done. For example, the Netherlands is one of the few countries to have quantified the relative significance of various loadings that influence the safety of buildings [6.2] [6.3]. The frequency with which incidents involving explosion and collision occur and their relative significance is documented and, presumably, well known. The general topic of dynamic problems associated with buildings is well covered in a study published by the Institute TNO for Building Materials and Building Construction [6.4]. Insofar as specific types of abnormal loadings are concerned the following are covered in some detail.

6.2 Gas Explosion

Subsequent to the Ronan Point failure a Committee (B16 of the Building Research Foundation) was set up to study and investigate the structural consequences of gas explosions in high-rise apartment buildings. The research and findings of this Committee are summarized in Report No. 29 of the Stichting Bouwresearch (Building Research Foundation) entitled: "Constructieve maatregelen tegen aardgasexplosies in hoge woongebouwen" ("Structural measures against natural-gas explosions in high-rise blocks of flats" [6.5]).

The recommendations of this Committee are particularly significant and may be briefly summarized as follows:

i) Domestic appliances and pipelines from which gas could inadvertently be allowed to escape should be made safe, and efforts to achieve this should be promoted.

ii) Rooms to which gas may have access should be adequately ventilated in order to obviate accumulation of gas (for a relatively slow rate of gas escape, anyway). Central heating boilers should preferably be located beside or on rather than inside the building. Gas supply pipelines to a building should be made flexible so as to avoid fracture due to differential settlement.

iii) The most effective procedure is to design the structure in such a way that, following the occurrence of local damage, the loadbearing function of a member which has failed can temporarily be performed by the rest of the structure (alternative path), so that progressive collapse is thus avoided. This means that the designer must take account of the pattern of forces (usually very much altered) after structural damage has taken place.

iv) The "alternative path" design principle often runs into practical difficulties. Besides, there remains the question as to what degree of damage would have to be taken into account. It is therefore recommended that the structural members be designed so that the risk of their failure in consequence of special loads is acceptably reduced. The phenomenon chosen, on more or less intuitive grounds, was the explosion of natural gas. This is the domestic gas used most widely in the Netherlands. The loading associated with a gas explosion is defined in Figure 6.1. It depends largely on the strength and size of the blow-out or venting wall areas, i.e., the relatively weak non-structural walls which may, and must, fail when an explosion occurs and whose failure will not produce serious consequences for the loadbearing structure.

6.3 Vehicular Collision

An excellent bulletin dealing with both vehicle loading and structural response has been published by the Spanbeton Company entitled "Botsingsbelasting" (Collision Loading) [6.6]. This publication is primarily directed at bridges. While these guidelines may not have the authority of a government publication they would be invaluable to any designer attempting to satisfy Clause 3.a. in the Dutch Building Regulations.

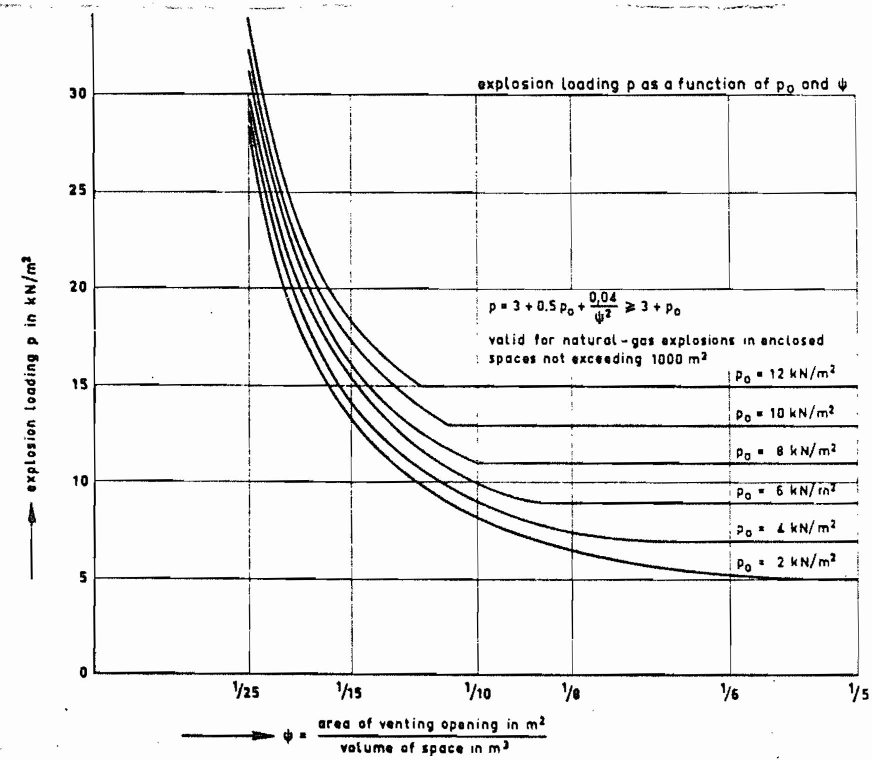

Figure 6.1 - Explosion Loading Pressure* Plotted as a Function of p_o and ψ. [6.5]

The meaning of the symbols employed in Figure 6.1 are as follows:

p = explosion loading, to be regarded as a uniformly distributed static loading on a structural member;

p_o = uniformly distributed static loading at which failure of the venting wall(s) occurs;

ψ = F/V = ratio of the area F(in m^2) of the venting wall(s) to the volume V(in m^3) of the room in which an explosion may occur.

* These are equivalent pressures as some provision for dynamic structural response has been made [6.5].

6.4 Summary

If it is assumed that in addition to NEN 3850 these documents on abnormal types of loading are representative of acceptable practice, then the Dutch approach could be summarized as follows:

- The building regulations merely enunciate the problem. Structural integrity is emphasized but neither guidelines nor minimum standards (e.g., minimum tie forces) are specified.
- The alternate path approach is advocated but the practical problems associated with this approach are recognized. Data on failure theories, catenary action, etc., do not appear to be available and it would appear that in most cases where abnormal loading is considered to be significant the element or system must be designed to accommodate equivalent explosive or impactive loads.
- An expression for explosive loading is provided. The development of this equivalent static loading and the argument for its use are very well presented in [6.5].
- Information is available relating to the equivalent vehicular loading and related structural response.

CHAPTER 6: REFERENCES

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[6.4] COMMISSIE VOOR UITVOERING VAN RESEARCH. Dynamische Problemen in de Bouw. CUR Report No. 57, Amsterdam, December 1972, 88 p.

[6.5] DRAGOSAVIC, M. "Structural Measures Against Natural-Gas Explosions in High-Rise Blocks of Flats", Heron, Vol. 19, No. 4, 1973, 51 p.

[6.6] DEKKER, A.J. "Botsingsbelasting", Nederlandse Spanbeton Maatschappij B.V., Bulletin No. 5.1.3, October, 1973, 20 p.

CHAPTER 7: CANADA

7.1 Introduction

In Canada the Building Research Division of the National Research Council has been notably active in dealing with problems related to progressive collapse. Records of structural failures are maintained and two of their reports are especially pertinent [7.1, 7.2]. In addition, this agency has largely been responsible for formulating the Code provisions on structural integrity first introduced in the 1970 National Building Code of Canada (NBC). A new edition of the NBC [7.3] is due to be issued in early 1975 superseding the 1970 edition. The NBC is truly a national code and as such is the single most important document relating to Canadian building practice.

Both the 1970 and 1975 editions approach the problem of the avoidance of progressive collapse in a similar manner. The following discussion will, however, be limited to the later edition. Article 4.1.1.8 entitled "Structural Integrity", which appears in the 1975 NBC, states that:

> *"Buildings and structural systems shall provide such structural integrity, strength, or other defences that the hazards associated with progressive collapse due to local failure caused by severe overloads or abnormal events not specifically covered in this section are reduced to a level commensurate with good engineering practice".*

Immediately following this statement is a note to the effect that reference should be made to Commentary C on "Progressive Collapse and Structural Integrity" [7.4] for further information. Apart from Article 4.1.1.8 and Commentary C there does not appear to be any other regulation that deals with the topics of abnormal loading or progressive collapse or any related aspect of structural integrity.

While Article 4.1.1.8 is couched in general terms it is very specific in intent and places all responsibility squarely, if not fairly, on the design engineer. It requires, somewhat optimistically, that the building be designed for the same level of risk (presumably risk to property as well as to life) irrespective of whether the building is subjected to normal or abnormal loading. It is difficult to evaluate levels of risk for normal, i.e., code specified, forms of loading. It is even more difficult to quantify "good engineering practice". Abnormal loadings, by definition, are not specified in the code and, by virtue of their abnormality, are not readily amenable to quantification. For these reasons, explicit compliance with Article 4.1.1.8 is difficult, if not impossible. Since Commentary C is the only regulatory assistance presently available it will undoubtedly determine Canadian practice with regard to abnormal loadings and their consequences and must therefore be examined in some detail.

7.2 Commentary C to the 1975 NBC

Because Canadian practice is in many respects similar to that in the U.S. the full text of Commentary C is appended (see Appendix C). The purpose of the Commentaries to the NBC is to provide both background and detailed design information and, in certain cases, to suggest design approaches. While the recommendations in the commentaries are not mandatory, their precise legal status is not very clear.

The Commentary is helpful in providing the following background information:

i) a list of the abnormal events that are referred to in Article 4.1.1.8. Unfortunately neither the nature, incidence or risk associated with any one of these phenomena are quantified.

ii) a list of 27 relevant references.

iii) a review of the historical precedents for considering progressive collapse and the current relevance of the problem to Canadian practice.

As far as design guidance is concerned the commentary discusses, in general terms and without quantification or numerical illustration, the following topics:

1. Ductility and its benefits are briefly mentioned. In reality the question of the nature, amount and distribution of reinforcement both

within and between structural elements is of crucial significance. Ductility is only one aspect of the detailing problem. Continuity, tie forces, catenary action, etc., are all interdependent considerations and, for design purposes, the coverage and guidance provided by the Commentary is less than adequate. Moreover, the Figure C-1 that is used to idealize and generalize various points could be misleading.

2. As far as designing for a specific abnormal loading is concerned, reference is made to the British Fifth Amendment [2.3] which requires that critical elements be able to withstand a 720 psf equivalent static overpressure in lieu of a gas loading. As the British regulations have been considerably revised since the Fifth Amendment was issued, the approach adopted in CP 110 [2.11] might well be preferred. Nevertheless is should be evident that two sentences in a commentary cannot adequately do justice to this particular topic. The mere fact that reference is made to British practice leaves the designer in the position of being unable to plead ignorance but without any meaningful assistance in meeting his professional and legal responsibilities. Presumably if the designer acknowledges that gas explosions are significant he is then under some obligation to design the building to accomodate this abnormal loading. He is not obliged to follow British practice. Nevertheless, because this approach is specifically mentioned, a standard is set for both the designers as well as the building authority. Given the economic realities of North American practice, the designer is likely to be under considerable pressure largely to ignore the problem of designing the structure to accommodate any particular abnormal loading.

3. Probably the most useful section of the Commentary is the discussion concerning floor layout and the prerequisites for a "good" floor plan. The advantages of spine or longitudinal bearing walls and returns are emphasized although the emphasis is somewhat weakened by the use of a non-typical illustrative example, Figure C-2. To deal with the behavior of the structure when a vertical element has notionally or otherwise been removed, it is mentioned that, with appropriate detailing, catenary action in the slabs and cantilever action in the walls can be utilized for the purposes of avoiding progressive collapse. Unfortunately, the

use of this alternative path approach and subsequent consideration of the damaged structure under large deformations begets more problems than can readily be resolved. For instance, most designers have had little or no experience of the performance of damaged buildings and insufficient data exist to exploit catenary action with any degree of confidence. Because no mention is made of any simple alternative, e.g. a set of minimum tie forces, the responsible designer is faced with a problem that he may not be equipped to solve properly.

7.3 Summary

The Canadian building regulations acknowledge that abnormal loadings occur and that structural failure, particularly progressive collapse, should be avoided. Formal recognition and definition of the problem in a code does, however, presuppose either its resolution or its solution. In North America this may not necessarily follow since the average designer and the industry has had little or no experience with:

i) considerations of abnormal forms of loading,

ii) the evaluation of risk or damage,

iii) design for explosive or impactive loading,

iv) the behavioral mechanics of damaged structures where deformations may be large,

v) debris loading.

Commentary C to the NBC attempts to provide background and conceptual advice. If the actual code provision were merely a cautionary statement the Commentary would perhaps be adequate but Article 4.1.1.8 is stated in positive, albeit general, terms. It is from this standpoint that the Canadian regulations have been examined. It is evident that, because of the nature of the problem and the intent of the code provision, it would be practically impossible to write an adequate Commentary. Moreover, the Commentary itself suffers from one serious flaw. By default, it is implied that the British regulations should be followed. This poses a dilemma for the designer that should have been resolved by the code writing agency. In brief, why should the designer be asked to absorb the cost of the design effort involved in order to solve a problem for which he may have no 'feeling', little sympathy or knowledge and which is likely to increase building costs? Given the subjective and ill-defined nature of the problem, it may be much more expedient for him to ignore or dismiss the issue completely. This, however, does leave the designer and probably the public in a rather vulnerable position.

The problem (economic, educational and professional) of dealing with abnormal loadings on buildings must be faced and at least the Canadian regulations do pose the problem. Commentary C is of value because of the dearth of other pertinent North American guidance and information. Moreover, a supplementary report is being prepared by Dr. D.A. Taylor of the Division of Building Research of the National Research Council and research adviser to the Code Committee. This report [7.5] is expected to enlarge considerably upon the Commentary and provide numerical examples of some of the techniques mentioned therein.

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COMMENTARY C

PROGRESSIVE COLLAPSE AND STRUCTURAL INTEGRITY

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COMMENTARY C

Progressive Collapse and Structural Integrity

Progressive collapse is the phenomenon in which the spread of an initial local failure from element to element eventually results in the collapse of a whole building or disproportionately large parts of it.

It is desirable to make a distinction between general and progressive collapse with regard to the consequences of the initial "local" damage. For example, the failure of a column in a 1-, 2-, 3-, or possibly even 4-column structure could be expected to precipitate general collapse because the local ruptured element is such a significant part of the total support of the structure at that level. Such structures or parts of structures are beyond the scope of the present provisions guarding against progressive collapse, although some of the requirements to ensure their safety might be the same as those suggested in this Commentary.

The present clause in Section 4.1 of the Code dealing with progressive collapse and structural integrity, the coherence of a structure which limits the spread of local collapse, is:

> 4.1.1.8—Buildings and structural systems shall provide such structural integrity, strength, or other defenses, that the hazards associated with progressive collapse due to local failure caused by severe overloads or abnormal events not specifically covered in this Section are reduced to a level commensurate with good engineering practice.

Abnormal Events

It is not possible to design structures for absolute safety, nor is it economical to design for foreseeable abnormal events unless they have a reasonable chance of occurrence. However, when an event is reasonably foreseeable, Clause 4.1.1.8 requires the design to incorporate sufficient structural integrity, strength or other defenses to minimize the probability of progressive collapse.

Some of the incidents causing abnormal loads might be: explosions due to gas, boiler failures, ignition of some industrial liquids or bombs; vehicle impact; falling or swinging objects, usually during construction or demolition; adjacent excavation or flooding causing severe local foundation failure; defects arising from extreme construction or design errors; very high winds such as tornadoes; and, sonic booms.

Most of the foregoing events would not be ordinary design considerations; however, events such as fires, earthquakes and corrosion, which the Code requires to be taken into account during the ordinary design process should also not cause progressive collapse at the specified load levels.

Major disaster can result from an incident where final damage in a structure without adequate integrity may be totally disproportionate to the initial local damage. A prominent case which focussed world attention on this problem was that of a 22-storey apartment block of large, precast concrete, load-bearing panels at Ronan Point,[1] Canning Town, England, in 1968, where a domestic gas explosion in an 18th storey apartment blew out the livingroom wall. The explosion led to the collapse of the whole corner of the building, when the apartments above, suddenly losing support from below, and being insufficiently tied and reinforced, collapsed one after the other. The falling debris ruptured successive floors and walls below the 18th storey and the failure progressed rapidly to ground level. Although no one was killed by the explosion, 4 persons died in the other apartments which collapsed.

After the tragedy a Commission of Inquiry investigated the incident and made a number of controversial recommendations,[1] some of which are discussed in this Commentary. Subsequently, further guidelines and discussions were published[2 to 9] to clarify the design problems and resolve doubts about the economic effects of the Commission's recommendations, and a great deal of research on the behaviour of building systems[10 to 16] and on the effects of gas explosions[17][18][19] was initiated.

Progressive collapse is being studied elsewhere in Europe,[20] the United States[21][22][23] and in Canada.[24][25][26] As an analysis of newspaper articles shows, 75 incidents were reported here in the 10 years from 1962 to 1972, of which almost 50 per cent occurred during construction. A well-known case[27] occurred in February 1959 in Listowel, Ontario, where the local arena collapsed under high snow loads during a hockey game, and resulted in 8 deaths and many injuries. Fracture of one of the laminated timber roof trusses led to a lateral progressive collapse of the whole roof and side walls.

Present Situation

The incidence of progressive collapse in Canada has, it seems, been acceptable; however, although it is difficult to determine whether the number of such incidents will be increasing it may be, for a number of reasons.

(1) There is a lack of awareness that structural integrity against progressive collapse is important enough to be regularly considered in design.
(2) The number of high-rise buildings with loadbearing walls is increasing.
(3) In attempting to achieve economy in building through greater speed of erection and less site labour, there is a tendency to build systems with the minimum of reinforcing and tying steel in walls, slabs and joints.
(4) In order to have more flexibility in floor plans and to keep costs down, internal walls and partitions are often non-loadbearing and hence are unable to assist in preventing progressive collapses.
(5) In roof trusses and arches there may not be sufficient strength to carry the extra loads, or sufficient diaphragm action to maintain lateral stability of the adjacent members if one collapses.

DESIGN CONSIDERATIONS FOR PREVENTING PROGRESSIVE COLLAPSE

To ensure structural integrity, good ductility and energy absorption capacity are desirable. Further, if the structure cannot be designed to bridge across missing elements, then these elements should be designed to remain functional under the abnormal conditions being considered. In cases of explosions, the risk can be mitigated somewhat by preventing the use of gas or storage of explosive materials, and the risk due to vehicle impact can be reduced by providing fenders. However, some measure of structural integrity should still be ensured.

While the following discussion applies primarily to buildings with loadbearing walls, precast column and beam structures and precast beam and floor slab systems should be considered in the same manner.

Ductility

Generally, connections between structural components should be ductile and capable of large deformations and energy absorption under the effect of abnormal conditions (see Figure C-1). Joints relying on friction due to gravity only are brittle in nature and their ultimate behaviour unpredictable, hence they are generally not adequate.

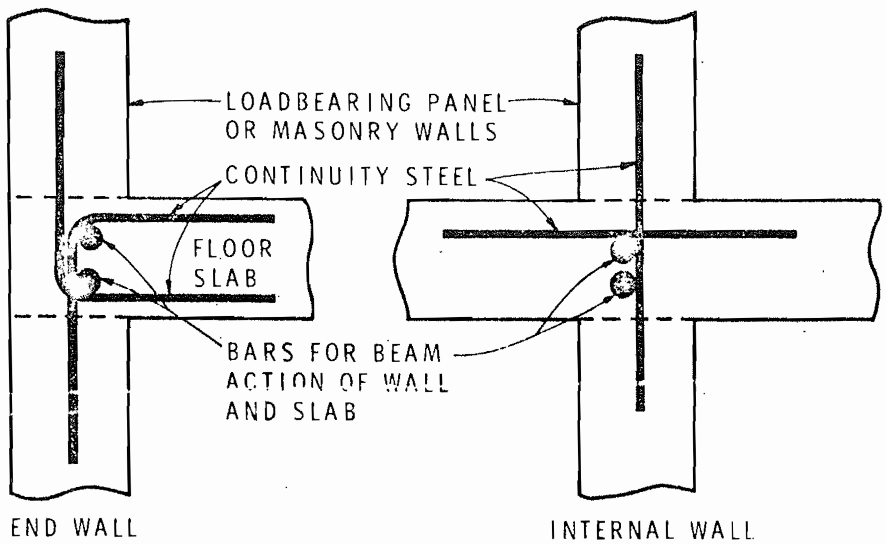

Figure C-1 Idealized example of ductile connections showing continuity steel; longitudinal bars providing cantilever and beam action in walls are also shown. (Diagrammatic only.)

Design for Abnormal Loads

If the removal of a structural member by a foreseeable abnormal event will initiate progressive collapse, the member should be designed to remain just functional under that condition. For example, in apartments containing gas appliances, British regulations[5] require design of the critical components for a 720 psf explosive pressure plus Dead Load + ⅓ Live Load + ⅓ Wind Load with a "safety" factor of 1.05.

Design for Alternate Paths by Which the Load May be Supported

Usually, the safest and most economical method of coping with progressive collapse is to design the structure in such a way that it can bridge the gap left when a structural component is removed. There are a number of ways of achieving the required integrity to carry the loads around missing walls, trusses, beams, columns and floors.[11]

Good Floor Plan. Probably the most important is the proper plan layout of walls (and columns). In bearing-wall buildings there should be an arrangement of longitudinal spine walls to support and reduce the span of long sections of cross-wall (see Figure C-2), thus enhancing the stability of individual walls and of the building as a whole. In the case of an explosion or vehicle impact, this will also decrease the length of wall likely to be affected. British regulations indicate that for properly designed and constructed loadbearing masonry it is unlikely that a length of wall more than 2¼ times the storey height will be blown out.[5]

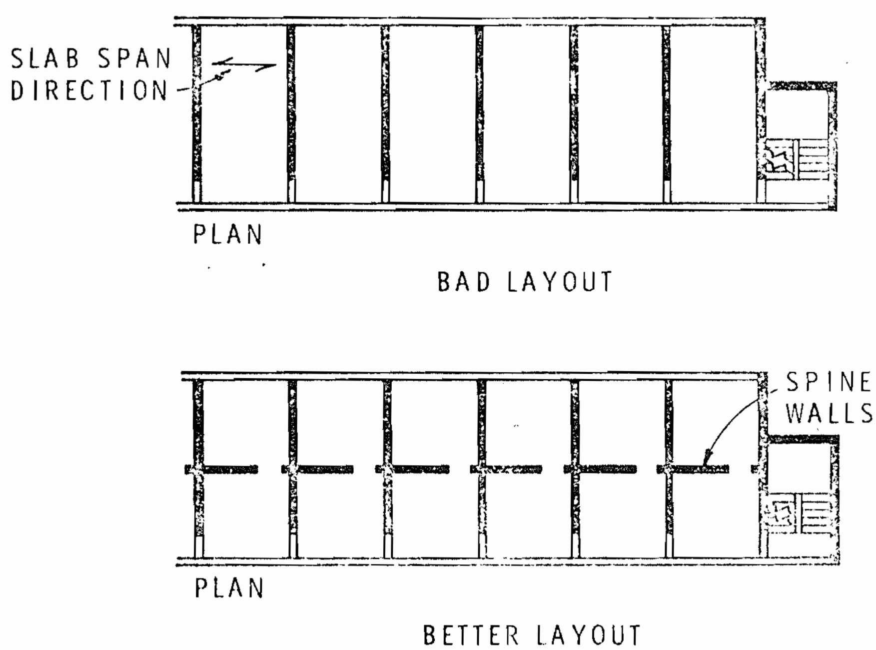

Figure C-2 Use of spine walls

Note: Only loadbearing walls (heavy lines) and external cladding are shown.

Return on Walls. Returns on internal and external walls will make them more stable.

Changing Direction of Span of Floor Slab. Where a floor slab is reinforced in order that it can with a safety factor of 1.05, for instance, span in another direction if a loadbearing wall is removed, the collapse of the slab will be prevented and the debris loading of other parts of the structures minimized. Often shrinkage, temperature and distribution steel will be enough to enable the slab to span in a new direction (see Figure C-3).

Loadbearing Internal Partitions. In order to achieve the change of span in the floor slabs, the internal walls must be capable of carrying enough load to support the edge of the slab as shown in Figure C-3.

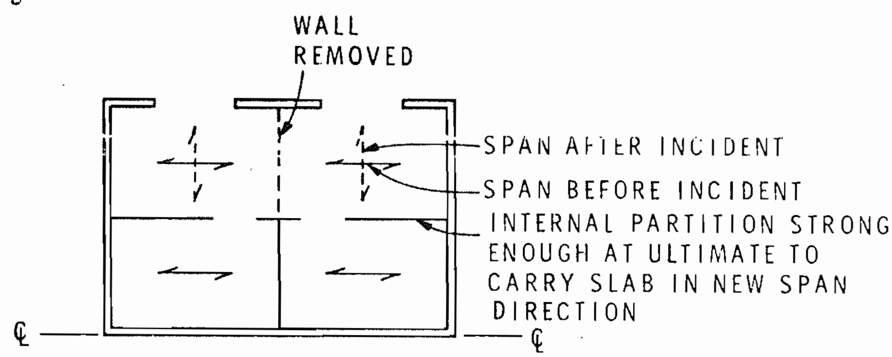

Figure C-3 Loadbearing internal partitions and change of slab span direction

Catenary Action of Floor Slab. Where the slab cannot change span direction, the span will increase if an intermediate supporting wall is removed. In this case if there is enough reinforcing throughout the slab and enough continuity and end restraint, the slab may be capable of carrying the loads by catenary action, though very large deflections will result.

Beam Action of Walls. Walls may be assumed capable of spanning over an opening if sufficient tying steel at the top and bottom of the walls (usually in the slab as shown in Figure C-1) allows the wall to act as the web of a beam with the slab above and below as flanges (Figure C-4).

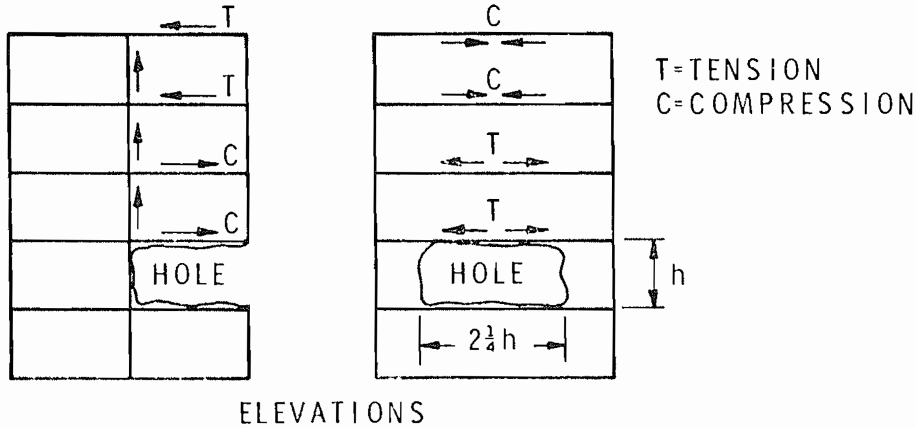

Figure C-4 Beam action of walls showing flange forces in floors

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CHAPTER 8: FRANCE

8.1 Introduction

Apparently the principal statement concerning safety that appears in a structurally oriented building regulation in France is the following:

Art. 10 - La construction doit être telle qu'elle résiste dans son ensemble et dans chacun de ses éléments à l'effet combine de son propre poids, des charges climatiques extrêmes et des surcharges correspondant à son usage normal.

This is taken from a document entitled "Construction - general regulations for the construction of residential buildings", No. 69-88 dated June, 1969 [8.1]. A translation is as follows:

Art. 10 - "Normally the entire building as well as each of its components must be able to withstand the combined effects of load due to climatic extremes and the overloads of normal usage".

Insofar as structural considerations are concerned the French building regulations do not appear to consider abnormal forms of loading. There appear to be no explicit recommendations or guidelines to accommodate local damage or avoid progressive failure. In the words of M. Kavyrchine of the Centre Experimental de Recherches et d'Etude du Batiment et des Travaux Publics, "The philosophy is that a building has to be well tied in all directions and some recommendations are given concerning the importance of ties (horizontal reinforcing bars at each floor level for instance); it is considered that a building designed following what we call 'Regles de l'Art' affords sufficient safety against abnormal loading" [8.2].

This situation is remarkable since the regulation quoted was issued well after Ronan Point and there is a considerable amount of industrialized building construction in France. Moreover the use of gas is widespread and in recent years there have been a number of spectacular explosions, e.g., at Argenteuil on 21 December, 1971, and at Auch on 4 January, 1971. Neither the C.E.B.T.P. (Centre Experimental de Recherches et d'Etudes du Batiment et des Travaux Publics) nor the C.S.T.B. (Centre

Scientifique et Technique du Batiment) are doing research directly related to abnormal loading or the avoidance of progressive collapse. Apparently there is little pressure, either governmental or professional, to initiate revisions to current structural design procedures.

From a purely structural viewpoint the French attitude to abnormal loadings and progressive collapse seems to be very different from that of most other countries in Western Europe. This is not to imply that the problems have been ignored, nor is it intended to suggest that the French accept a higher level of risk in building. The situation in France is unique and considerations of structural safefy must be examined in a broader context than that provided by the formal regulations alone.

8.2 Public and Professional Awareness

The Ronan Point disaster occurred on 16th May, 1968. It is generally believed that this incident had international impact on both the public and the design profession because of the extensive news coverage it received. This was not the case in France which, at precisely this period, was undergoing both student and labor unrest if not revolt. France was virtually cut off from the outside world at that time and its economic life paralyzed. Under the circumstances it is unlikely that the Ronan Point incident attracted much public attention. In due course this incident and related issues did receive professional consideration and the three papers by Robinson [8.3], Despeyroux [8.4] and Saillard [8.5], published in November 1969, provide an excellent review of French attitudes and practice which, for the most part, still prevail.

8.3 Gas Usage

There are non-structural procedures for reducing risk due to abnormal loadings. For example one could reduce the probability of occurrence to a suitably low level. This, in fact, is what has been done in France with regard to explosions from domestic gas, probably the most prevalent of all forms of structurally significant abnormal loading.

Regulations governing the distribution, installation and use of piped gas are much more restrictive in France than say in either the U.S., Canada, or Britain. For example the fire regulation 67.216 [8.6] states that in high-rise buildings (higher than 92 feet, 9 to 10 floors, for all buildings except those used solely for residential purposes where the limiting height is 166 feet) the storage and use of gas (as well as other combustible liquids or solids) is forbidden. If a gas boiler is to be used it must be located on the roof of the building and, in addition, all gas lines must be external to the building. In Article GH7 of these regulations it is stated that provision must be made for explosion if it can occur. In non high-rise buildings the regulations do permit gas supply lines in the building but, especially if a high pressure supply is used, these must be enclosed in vented shafts [8.7]. Apparently as a consequence of the explosion in a 14-story building at Argenteuil the regulations concerning the use of gas have been amended [8.8] and it would appear that, subsequent to 1 July, 1972, all apartment buildings (batiment d'habitation) with gas boilers must have them mounted on the roof with the gas supply lines mounted exterior to the building. Without going into too much detail it is evident that both gas and fire related regulations have obviated the need for major revisions to a structural code. Figures for gas explosion frequency or the acceptable level of related risk are not available but, as evidenced by the 1972 amendment [8.8], the French have chosen to manipulate the load side rather than the response side of the design equation.

8.4 Nature of the Regulatory System

Another unique aspect of French practice is the number and nature of agencies involved in the regulation of building design, construction and usage, especially where industrialization or innovation is concerned. For example the insurance business is an active and important component in the building control process. Large companies, e.g., Socotec, not only cover financial risk but are also directly involved in the technical monitoring of both design and construction. To do so they must provide specialist consulting service and a high level of supervision. Thus a non-governmental agency with direct financial authority and responsibility can and does have a real influence on the building components and systems. The centralization of research effort, at C.S.T.B. and C.E.B.T.P. for example, the emphasis on development, plus the existence of the C.E.B. provide a professional environment that, if not unique, is rather different from that in North America. While the C.E.B. has no governmental or formal authority it undoubtedly has considerable influence and it may be of interest to document their activities with regard to abnormal loadings and progressive collapse.

8.5 Comité Européen du Béton (C.E.B.)

As early as July, 1967, the C.E.B. published recommendations for the design and construction of large-panel structures [8.9]. In emphasis and choice of language these recommendations had and still have considerable merit and the following items are particularly pertinent:

i) Overall Considerations: In the introduction to the recommendations explicit mention is made of the desirability of avoiding progressive collapse (in their words, the "house of cards" effect). Reference is also made to the need for mechanically continuous ties and the fact that, in a panelized structure at or close to failure, there is little possibility of a redistribution of forces.

ii) Loading: Apart from a reference to exceptional errors of execution (R73.4) there is no attempt to identify or specify any abnormal loading. However there is a requirement (R22) that all members be designed to accomodate a horizontal force of 1% of the load* on the member

iii) Horizontal Joints and Ties: Section R14 requires that:

> Within the thickness of each floor, or close to the floor, mechanically continuous steel "ties" should be provided in both directions; these ties should interconnect the walls or facades on opposite sides of the building, should include all the vertical panels, and should be connected to each panel.

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* In the English translation the term 'weight' is used but this would appear to be an interpretive error.

Peripheral Ties: The total cross-sectional area of the longitudinal reinforcement provided over a story height in a peripheral wall must not be less than 2 cm^2 [0.31 ins^2] irrespective of the grade (strength or class) of steel employed.

Internal Ties: The total cross-sectional area of the tie reinforcement inter-connecting two opposite external walls should be able to absorb a tensile force equal to 1% of the direct force acting (at the level considered) on the external wall in question, and equal to not less than 500 kg. per metre (336 pounds per foot) of external wall. This cross-sectional area may be concentrated at the cross-walls or it may be distributed in the floors.

Well before the Ronan Point incident a set of recommendations were available which made specific mention of progressive collapse and, albeit in an indirect way, made some provisions for the structure to be laid out and detailed to avoid this sort of failure. Saillard [8.5] commented that the Ronan Point building did not meet the C.E.B. recommendations and also stated that detailed examination of the British incident did not result in any changes in the C.E.B. document. While this last statement is indeed true, it is significant that in 1971 an Appendix [8.10] to the recommendations was prepared and published but neither accepted nor adopted by the C.E.B. Despite being stillborn this appendix is significant in that:

i) specific reference is made to abnormal loadings. In particular gas explosions and vehicle impact are identified and mention is made of the type of initial damage likely to be caused.

ii) without any possible ambiguity it is required that both vertical and horizontal tie forces of 8000 N/m (548 pounds per foot) be provided. This force, while somewhat larger than originally recommended, is not particularly onerous even when designed on the basis of service stress values.

Evidently the major intention of this appendix was to identify specifically the causes and delineate the issues involved without necessarily altering the intent of the actual recommendations. However by spelling out its nature the structural problem and the attendant professional responsibilities are given a relative significance that may not be warranted. Given the fact that in France the probability of a structurally significant gas explosion is relatively low and that, to date, no comprehensive abnormal loading surveys have been done, it is not too surprising that the original C.E.B. recommendations were considered to be sufficient.

8.6 Summary

At an official level there appear to be few, if any, regulations that concern the structural consequences of abnormal loading and the avoidance of progressive collapse. The need for structural regulation is largely obviated by the existence of restrictive regulations governing the installation and utilization of gas in buildings. Moreover, some fairly comprehensive recommendations for the design of large panel structures do exist. While there does appear to be some consensus to the effect that the British over-reacted to the Ronan Point incident [8.3], [8.5], there is apparently some support (as evidenced by the tentative Appendix to the C.E.B. Recommendations) for a more detailed and comprehensive treatment of the whole question of abnormal loadings.

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CHAPTER 9: EASTERN EUROPE

9.1 Introduction

The nature of the building process and its product in Eastern Europe is very different from that in North America. Nevertheless the intent of those regulations that govern the design and construction of buildings is of interest and in this chapter an attempt will be made to review certain building regulations in both Poland and Czechoslovakia insofar as they pertain to abnormal loadings or progressive collapse. Relevant extracts from the 1970 Comecon recommendations will also be considered.

No attempt will be made to evaluate considerations of implementation or practice with regard to these regulations. In each case, the recommendations involved relate specifically to panelized concrete construction. Therefore the perspective of this discussion is somewhat limited and the situation with regard to other building materials or types of construction is not considered. To do this adequately and comprehensively (i.e., include the U.S.S.R.) is beyond the scope of this particular study.

9.2 Poland

Dr. Bohdan Lewicki and his colleagues at the Center for Building Systems Research and Development in Warsaw have done and are currently involved in a number of studies concerned with progressive collapse [9.1], [9.2]. In addition, Dr. Lewicki, as Chairman of CIB Working Commission 23A on load bearing walls, has been instrumental in having published a policy document on limiting progressive collapse [9.3]. It would appear [9.4] that informed opinion in Poland is largely in agreement with this CIB document. To obtain some idea of the nature of the regulations consider the following pertinent extracts* from the code of practice for the design and construction of Large Panel systems [9.5].

* The author is indebted to Dr. S. Zieleniewski for this translation but has taken the liberty of altering some of his wording.

All structural elements that contribute to the stability and structural integrity of the structure in service shall be interconnected in order to ensure their interaction and their advantageous response to both horizontal and vertical loads. Joints should not increase the risk of progressive collapse due to gas explosion, vehicular impact, etc.

Those floor slabs supported by the walls of the topmost story shall be connected either directly or by means of a tie beam with these vertical wall panels. The connection shall be able to resist vertical forces applied to the underside of the floor [roof] slab e.g., caused by a gas explosion. The amount of tensile reinforcement across such a joint shall not be less than 1 cm^2 per metre [0.47 ins^2 per foot] of wall support.*

The interconnections of floor slabs at a support shall permit the transference of tensile forces and prevent and protect against the sudden collapse of a floor element if, for example, a support wall were to be removed as a consequence of some exceptional action. In addition, to increase the flexural stiffness of the floor slab and to ensure the uniform transfer of vertical load the interconnection of two floor panels across the support should be designed for negative moment. Such a connection is strongly recommended when lightweight concrete wall panels are used.

When monolithic tie beams are used they shall be connected to both wall and floor elements. These connections shall limit the possibility of sudden collapse of floor elements under exceptional loads. Moreover they shall limit the possibility of the extension or spread of failure by creating favorable conditions for the external wall element to behave as a cantilever [see Figure 9.1].

The vertical reinforcement joining structural walls shall also be anchored in these tie beams.

The tie beams should be designed so that when subject to a catastrophic loading the structure performs in a manner similar to that of a monolithic [cast-in-place] building.

The cross-sectional area of tie-beam reinforcement i.e., F_{zw} irrespective of the type of steel, shall meet the following conditions:

- $F_{zw} \geq 2.3\ cm^2$ [0.357 ins^2] *when the floor slabs are supported along three of four edges i.e., reinforced in two directions, or along two edges if the length of wall panel along the tie-beam is not greater than 4.80 m. [15.75 ft.].*

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* Items in square brackets i.e., [ ], have been inserted by the author.

- *$F_{zw} \geq 3.4$ cm^2 [0.527 ins^2] when floor slabs are supported along two edges and the span of floor slabs along the tie beam is greater than 4.80 m [15.75 ft.] but not greater than 6 m. [19.7 ft.].*

- *$F_{zw} \geq 0.08L$ cm^2 [.0038L ins^2] where L is the length in metres [feet] of the tie beam under consideration.**

When the floor slabs are interconnected to take negative moment moment over a support, F_{zw} in the tie-beam parallel to the span of the floor slabs shall not be less than 2.3 cm^2 [0.357 ins^2] independent of the value of L.

The approach could be summarized as follows:

- identification but without quantification, of the problem. In this case the solution is to reduce the probability of progressive collapse due to abnormal loads.

- the use of tie beams is encouraged.

One criticism is that two distinct issues appear to be confused, namely, the avoidance of progressive collapse irrespective of type of loading, (i.e., normal and abnormal) and the accommodation of abnormal loadings. The wording of the regulation apparently reduces the objective to avoiding progressive collapse due to abnormal loading only.

There are four aspects which are of particular interest:

i) the explicit mention of the fact that uplift can occur in the topmost story. The amount of steel required (of the order of #3 at 24 inches) is not onerous but the obligation to provide this connection is significant.

ii) particular attention is given to the connection of floor elements at a support in an attempt to avoid dislodging floor panels.

iii) the use of tie beams, hidden or otherwise, is dealt with somewhat more explicitly than in other codes. These effectively constitute the peripheral and internal ties in the horizontal direction and, given the amounts of steel specified, the tie force (on a per width or span of slab basis) is relatively low and comparable to that required by the CEB recommendations [8.9].

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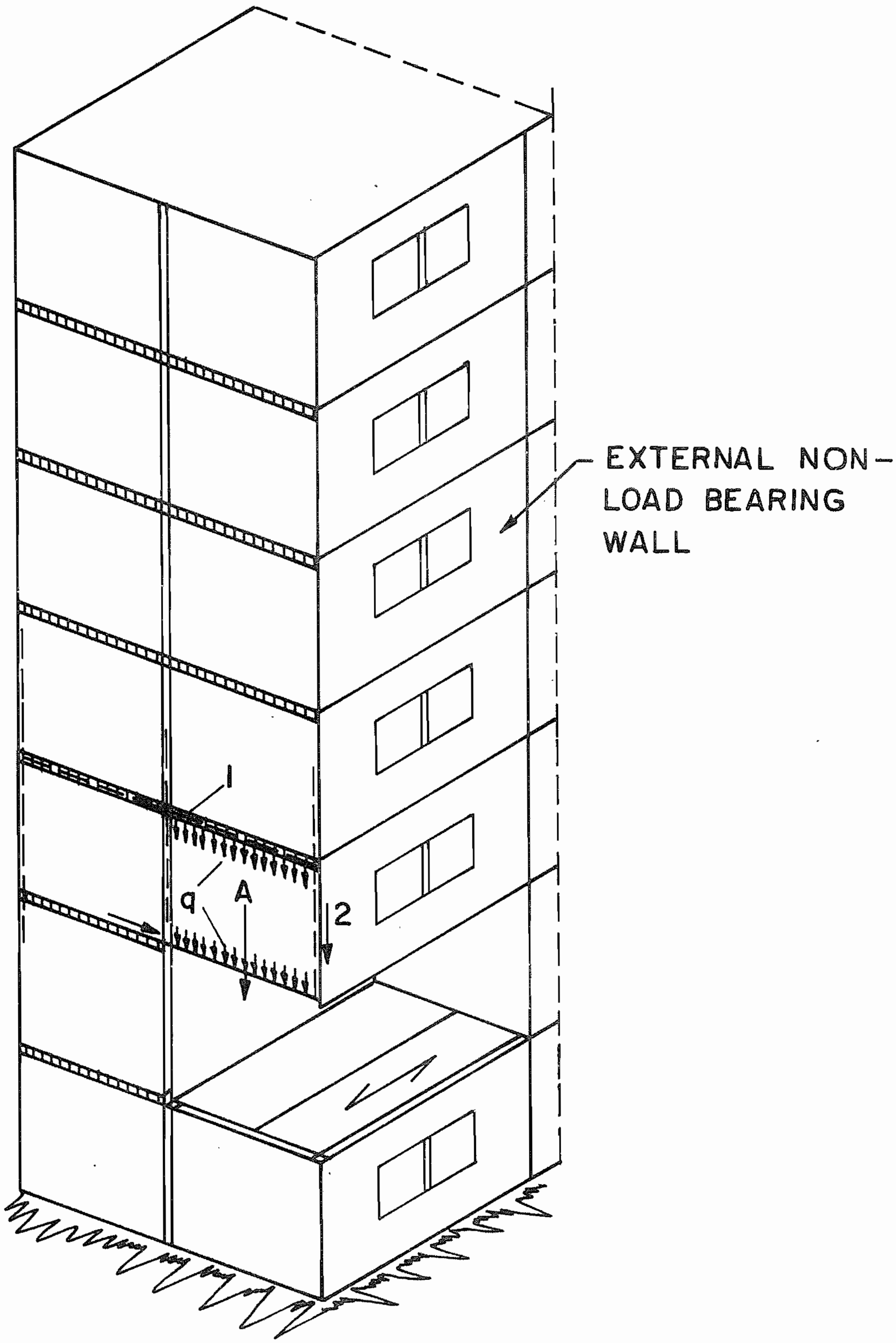
* It would appear that either .8L or .08L^2 would be a more appropriate value and, pending clarification by Dr. Zieleniewski, this value should be treated with caution.



q - LOAD FROM THE FLOOR SLABS
1 - RESPONSE OF TIE BEAM
2 - DEAD LOAD OF WALL PANEL

Fig. 9.1 DAMAGED SYSTEM RESPONSE

iv) the concept of removal of at least one external room-sized vertical element and the development of alternative support paths is raised (see Figure 9.1) but, apparently, as a behavioral model and not as a specific design requirement or option.

9.3 Czechoslovakia

Large panel construction is fairly common in Czechoslovakia and it is understood that a study of abnormal loadings and their consequences is currently being undertaken. In a recent text "Design of Joints in Panel Buildings; Volume 1 - Load-Bearing Joints" by Pume and Witzany [9.6], considerable attention is given to the topic of progressive collapse and various codes of practice are summarized. It is of interest to consider those Czech regulations that relate either to abnormal loadings or to progressive collapse. The following is a translation of pertinent sections of the "Recommendations for the structural design of panel buildings" dated 1970 and edited by VUPS (the Research Institute for Building Construction) [9.7]:

Other extraordinary loads

2.20 *The effects of extraordinary loads, the character of which may be unfavorable, (e.g., gas explosions in rooms, air pressures caused by airplanes, impacts due to heavy vehicles, local fires) must be prevented by means of certain structural (assembling) details.*

2.21 *If the assembling reinforcement, connecting the panels, is arranged according to Chapter IV of Recommendations [see below]* it is not necessary to calculate the effects of extraordinary loads.*

Horizontal stiffening of the building

4.5 *In the plane of every floor longitudinal and transverse reinforcing ties must be arranged for the purpose of stiffening the floor structure, and in addition, reinforcing ties must connect the load bearing walls together with the floor structure at its perimeter in accordance with Paragraphs 4.6 to 4.9.*

4.6 *Reinforced tie-beams are obligatory in the region of each intersection of the bearing wall and the floor. The reinforcement of the tie beam is to be designed [at service levels] for a tensile force equal to 1.5 ℓ/Mp/, where ℓ is the distance between the bearing walls, expressed in m. [ℓ kips for ℓ in ft.].*

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* Items in square brackets i.e., [ ], have been inserted by the author.

4.7 *The ties connecting the opposite outer bearing walls (gable walls, staircase walls, etc.) must be arranged parallel to the span of the floor panels. The tie reinforcement is to be designed for a tensile force [at service levels] equal to 1.5 Mp per 1 m. [1 kip per foot] of the width of the floor, provided that a larger force is not otherwise required.*

The distance between the ties must not exceed 120 cm. [4 feet]. Tie bars are to be inserted into either the vertical joint between the floor panels or in the floor panels.

If the bearing wall provided with the reinforcement in accordance to Para. 4.6 is situated in the direction of the span of the floor panels, one is allowed to consider this requirement to be part of the ties required in para. 4.7 within the 120 cm width.

4.8 *If the directions of the spans of the adjacent floor panels are perpendicular to each other, the reinforcement parallel to the extended over two bays and connected with the reinforcement required by Para. 4.6.*

4.9 *The connections between the outer bearing walls and floors must withstand the forces specified in Para. 4.7.*

Vertical stiffening of the building

4.10 *It is recommended that vertical reinforcement be inserted into the vertical joints between the wall panels of the load bearing walls. The cross-sectional area of this reinforcement is to be designed for a tensile force equal to the weight of the story height panel and this tie should be continuous across the joint.*

Without mentioning progressive collapse, the intent of these clauses is to accommodate abnormal or extraordinary loadings in general. Paragraph 2.21 permits and, since no loading is actually quantified, ensures that the indirect solution procedure implied by paragraphs 4.5 to 4.10, is followed. These effectively specify the location and amount of tie reinforcement required. The formation of peripheral and internal tie beams is emphasized. Tie force requirements are not particularly onerous. In common with both the CEB and Polish requirements tie forces are based on service load as opposed to "ultimate" or elastic limit considerations. It should be noted that the vertical tie requirement (para. 4.10) is not mandatory and that the tie force is presumably required to support a freely hanging wall panel in the event of some structural damage.

9.4 Comecon Recommendations

The following extract* from a "Draft recommendation for the choice of design schemes for large panel buildings in Socialistic Countries" [9.8] is reproduced in full.

6. Design Requirements for Accidental Loads

6.1 In the design of multi-story large panel buildings in which the possibility of an accidental explosion (gas or other explosive substances) is not excluded, it is not necessary to calculate all elements of the building for the direct load of an explosive wave. Instead, for premises in which explosions have some chance of occurring, the possible collapse of separate elements of walls (see Para. 6.2) is assumed, provided further collapse of the remaining building structure is not allowed. In this case significant deformation and cracks affecting the serviceability of the building can be allowed in the remaining structure.

6.2 As a result of an explosion inside of one of the premises of the building, the following collapse is conditionally assumed:

a) one panel of an inner cross wall of the premises, not adjoining the end of the building, or,

b) one panel of the external end wall.

For the collapse scheme of panels indicated above, the remaining structure of adjacent walls and floors must be calculated under only the vertical loads from dead weight and live loads on floors, without overload coefficients. In this case the limiting values of material resistances of walls (for concrete - design prismatic strength; for reinforcing - yield point) must be used for calculation. Redistribution of loads between the remaining elements of the building should be considered in the calculation, as well as possible plastic deformations in the panel reinforcing and connections as well as formation of cracks in the concrete.

6.3 To prevent progressive collapse of the structure of large-panel buildings as a result of local explosions and other unforeseen loads, besides the calculation requirements in Para. 6.2 the following should be considered in choosing structural solutions for the building:

a) the general arrangement of components of the building must provide the strength, general stability and spatial immovability of the building in case of local collapse of separate load-bearing elements. It is preferable to have a cross-

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* I am indebted to Dr. David Allen of the National Research Council of Canada for this document.

system of walls, including transverse load-bearing walls, with a closed cellular plan and slab floors supported along the perimeter. Structural building schemes consisting of longitudinal load-bearing walls and floors in the form of planking supported along two sides, are not recommended.

b) *prefabricated elements of floors must be safely interconnected by the use of welding of reinforcing continuity bars, by the use of continuous ties along the joints so as to create slabs which allow continuous horizontal redistribution of forces between the walls in failing situations.*

c) *in considering explosive loads, the most disadvantageous conditions can be shown to be the end walls; to increase overall stability of end walls they should be safety connected to the floors and adjoining longitudinal walls.*

d) *when non-load-bearing longitudinal external walls are used, they should be in the form of light hinged panels;*

e) *structures of load-bearing elements, as well as their connections, must be capable of developing plastic deformation under the action of random short-term loads (explosion-type) thus increasing their resistance to these kinds of loads.*

It is preferable to have continuous connections, without stress concentrations and not susceptible to brittle failure.*

These clauses should not be taken to be representative of the regulations or practice in the U.S.S.R. or any other country in Eastern Europe. Regardless of the probably 'academic' nature of these regulations, they are of considerable interest because of their comprehensive nature and the emphasis and priority given the various issues involved. The following comments are pertinent.

i) The use of terms such as 'accidental loads', 'local explosion' and 'unforeseen loads', is indicative of the problem of identification and classification that exists in dealing with what, in this study, have been collectively called 'abnormal loadings'. Nonetheless explosions (gas or otherwise) have been identified as being an important, if not the most important, form of abnormal loading in multi-story large-panel buildings.

ii) At the outset direct design to withstand the explosive wave is mentioned. Alternatively and preferably the "alternative path" approach (notional removal of any single vertical wall element) is recommended. It is rather conservatively suggested that the notionally damaged structure withstand the full, i.e., service level, normal dead and live loads, i.e.,

*Apparently the connection should be completely filled with infill concrete or mortar.

verticle, but not wind loads.

iii) Significantly the design choices are either to design all elements and the structural system to withstand the loading or, if this is not reasonable, to ensure that any element likely to fail under the explosive load, can fail (i.e., be rendered inoperative) without progressive failure of the remainder of the structure.

iv) Section 6.3 merely states a number of guidelines to be followed in addition to complying with Sections 6.1 and 6.2. The guidelines are of a non-quantitative nature and refer to layout and connection, i.e., tie requirements.

What, of course, is significant is that these general specifications recommend an approach somewhat different from that actually adopted in Poland and Czechoslovakia and recommended by the CIB Commission 23A. Perhaps even more significant is the recommendation in Clause 6.3(a) to avoid using building schemes consisting of longitudinal load-bearing walls and floors in the form of planking supported along two sides, i.e., typical North American residential construction practice.

CHAPTER 9: REFERENCES

[9.1] LEWICKI, B., K. DEUAR, and S. ZIELENIEWSKI, "Interaction of floor and wall as prevention of Progressive Collapse in Large Panel Buildings". Working paper presented at Meeting of C.I.B. Commission 23A in Copenhagen, September, 1973.

[9.2] LEWICKI, B., "Prevention of Progressive Collapse". Béton en Béton-constructuies/De Ingenieur, Vol. 83, No. 6, 12 February 1971, pp. 7-11.

[9.3] LEWICKI, B., and S.O. Olesen, "Limiting the possibility of Progressive Collapse". Building Research and Practice, Jan./Feb. 1974, pp. 10-13.

[9.4] Private Communication (October 10th, 1974) from Dr. S. Zieleniewski of the Centre for Building Systems Research and Development, Warsaw.

[9.5] MINISTRY OF CONSTRUCTION, WARSAW. "The Design and Construction of Buildings made from Large Panel Prefabricated Elements", Polish Building Standard, 1st January, 1975. (This standard apparently updates the previous standard PN/B-03253 issued in 1972).

[9.6] PUME, D., and J. WITZANY, "Navrhovani Styku Panelovych Konstrukci. 1. dil: Nosne styky". Vydavatolstvi CVUT, Praha, 1974, p. 152.

[9.7] VUPS (Research Institute for Building Construction) "Recommendations on the Structural Design of Panel Building", Prague, 1970.

[9.8] Committee for Economic Mutual Aid, "Draft design recommendations for large panel buildings in Socialistic Countries - Choice of design schemes", Moscow, Committee for Economic Mutual Aid, February, 1970.

CHAPTER 10: REGULATION, DESIGN AND SYNTHESIS

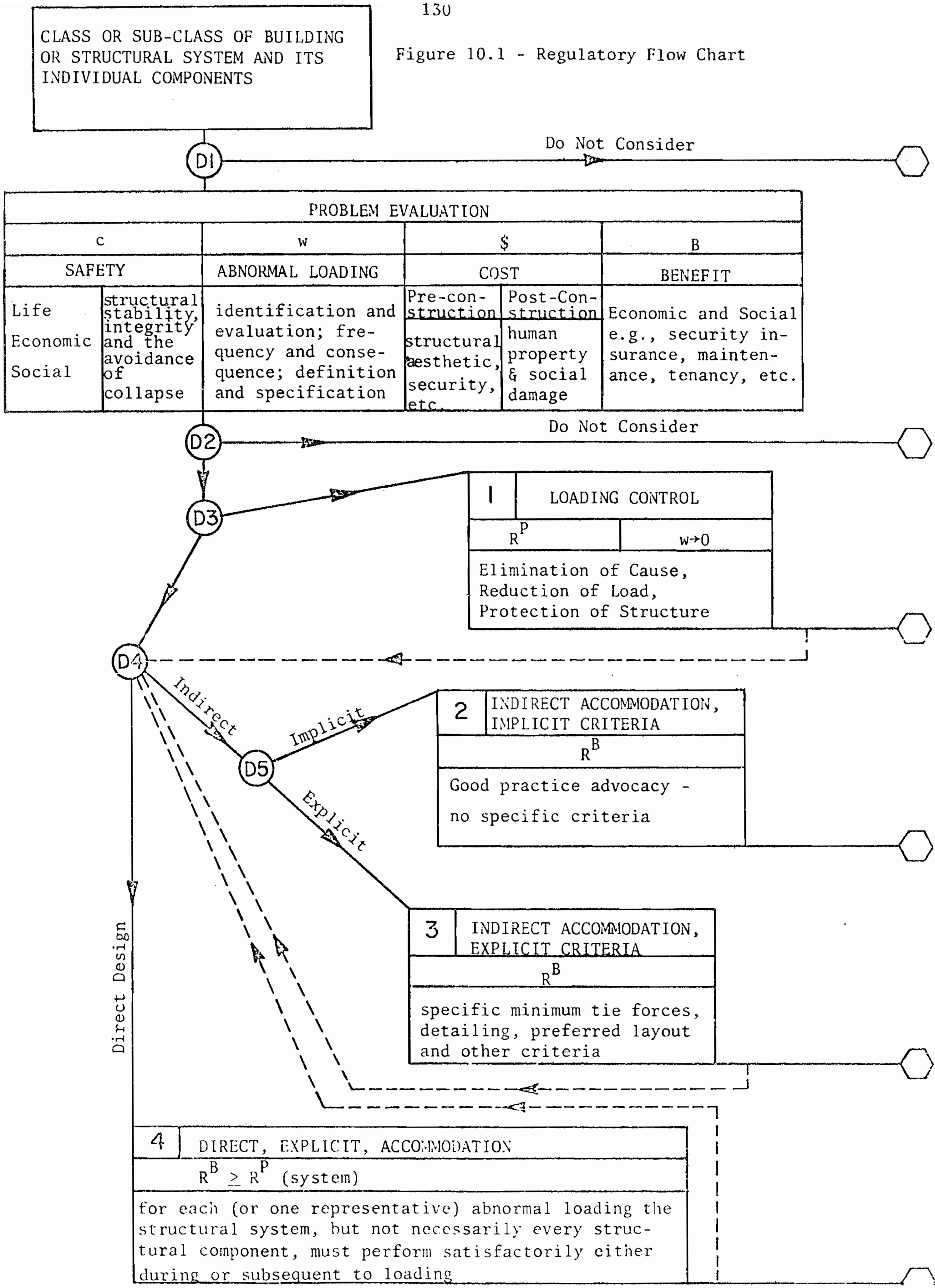
10.1 The Regulatory Problem and the Design Process

There are at present no generally accepted building regulations or design guidelines in the U.S. for the avoidance of progressive collapse or resistance to abnormal loadings. While the responsible structural designer may acknowledge and even attempt to resolve problems associated with abnormal loading, the primary initiative and responsibility for definition, quantification and guidance belongs to those groups or agencies charged with drafting codes of practice and building regulations. These groups are usually committees with members drawn from the design profession, the construction industry, government agencies and universities. Regulatory provisions dealing with abnormal loadings must satisfy two sets of criteria: those related to structural performance, particularly safety considerations, and those reflecting the economic, political and procedural realities of the building business. The latter are usually resolved by consensus but, in order to provide a framework for constructive discussion, the issue of structural safety has to be clearly delineated and, at the very least, qualitatively understood.

This study has set out to evaluate pertinent non U.S. building regulations. While there may be significant differences in the approach and intent of the building regulations in each country it is possible to generalize the problem facing any regulatory body by means of a flow chart identifying all the significant decisions, stages and alternatives involved. The flow chart in Figure 10.1 attempts to illustrate this common problem.

The structural designer is faced with a dilemma similar to that confronting the regulatory agency. In the absence of regulation he is professionally responsible for pursuing the solution procedure shown in Figure 10.2, the Design Flow Chart. This is very similar to the flow chart in Figure 10.1. Because of this procedural similarity the following discussion will attempt to cover aspects of implementation and compliance, i.e., the design viewpoint, in addition to those of regulation. The terms used to categorize the structural design process in Chapter 1

Figure 10.1 - Regulatory Flow Chart



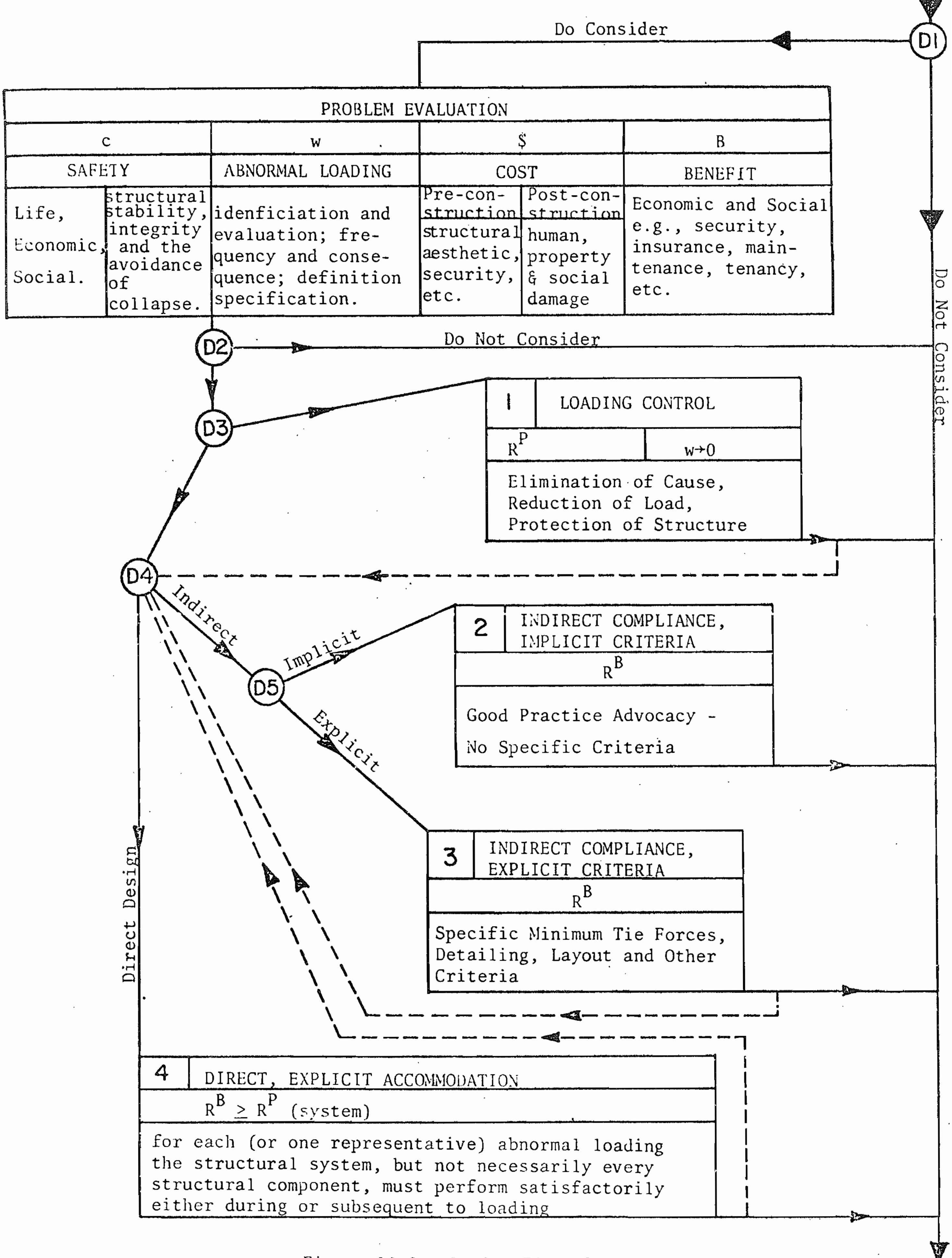


Figure 10.2 - Design Flow Chart

are also employed in this attempt to generalize the abnormal loading problem.

Unlike the designer, who is usually involved with only one building at a time, a regulatory agency will be concerned with classes or sub-classes of buildings. For example, low-rise, multi-unit residential; high-rise, multi-unit residential and high-rise commercial are three sub-classes of building for which the consequence of an abnormal loading may be significant. Whatever the sub-class of building, it is necessary that consideration be given to both the structural system as well as all individual components. Similarly in design the structural integrity of structural components, sub-systems and the overall structural system must be ensured.

The decision must be made whether or not to consider abnormal forms of loading, i.e., D1 in Figures 10.1 and 10.2. The regulatory agency must make this crucial decision. There are only two possible arguments to justify a decision to ignore all forms of abnormal loading and their consequences. These are:

i) convincing statistical and economic grounds for doing so. Existing data, although limited, do suggest that relative to normal forms of loading, abnormal loading requires design consideration.

ii) given the possible economic, professional and political consequences of any regulatory action it could be argued that the possible costs would outweigh the probable benefits and that the existing levels of risk are socially acceptable. An extension of this argument is the possibility that one sector of the industry may be inequitably affected. Given the fragmented nature of the building industry and the procedural, behavioral and other differences that exist in the clay masonry, concrete masonry and structural concrete sectors of the industry, it is possible that any material or product oriented regulatory action might initially penalize the more responsive and progressive sector.

10.1.1 Problem Evaluation

Having decided, i.e., D1 in Figures 10.1 and 10.2, that abnormal loadings are to be considered, it follows that the extent and nature of the problems, if any, should be evaluated as comprehensively as possible. The lack of reliable data and experience may compel the designer to accept a qualitative evaluation. However to determine and justify any action by a regulatory agency it is essential that, for each sub-class of buildings, serious consideration be given to quantifying and evaluating the following:

i) Structural safety insofar as this performance criterion involves human, economic and social considerations. The cost of death and injury as well as short term economic losses are to some extent measurable. The building engineering profession may not be accustomed to collecting and evaluating information of this nature, nevertheless these factors must be considered in conjunction with the structural integrity of the building.

ii) For each of the abnormal loadings listed in Table 10.1, the frequency of occurrence, structural consequences, and related probabilities should be ascertained. Much work on loadings, both normal and abnormal, has been done in recent years. Although much remains to be done there is sufficient information available to permit comparative decision making and the definition and specification of various loadings. One very useful service that a regulatory body could perform would be to provide encouragement, incentive and direction for future work in this area.

iii) In evaluating the cost aspects of the risk-cost-benefit tradeoffs both the pre-construction and the post-construction costs must be considered. The latter are costs incurred by an abnormal loading incident, i.e., human, property and possibly social damage. Pre-construction costs are considered to be those additional costs involved in designing buildings to avoid progressive collapse and to accommodate abnormal loadings.

iv) The measurement of benefit is relatively difficult and somewhat subjective. To justify whatever regulatory action is taken there must be some indication that the relative level of risk as a consequence of abnormal loading is reduced, made comparable to or made less than that due to normal loading.

Having defined and evaluated the overall problem it is then possible to make a rational decision as to whether abnormal loadings constitute a design problem or not, i.e., D2. Structural provision against abnormal loadings may be unnecessary, for example:

GENERAL TYPE OF ABNORMAL LOADING	SUB-CATEGORY	SPECIFIC EXAMPLES
Explosive	Solid, Liquid or Gaseous Explosive Material	natural gas (service system) steam, TNT, dynamite liquefied natural gas, propane, oxy-acetylene, etc.
Impactive	Vehicular, Missile, or Aircraft	car, truck, or other vehicle, crane accident, wind borne debris, vandalism, etc.
Static	Gravity or Hydrostatic	flooding, (i.e., service system malfunction), debris loading, etc.

Table 10.1 - Identification of Abnormal Loadings

i) in those instances where (because of geographical location or occupational hazard) the severity of "normal" loadings such as earthquake, wind, blast etc., provides the building with sufficient strength and deformability presumably to withstand or accommodate an abnormal loading.

ii) in those instances where, irrespective of the level of risk, structural provisions to avoid collapse or failure are not economically feasible etc., single family dwellings; small, non-engineered buildings, etc.

10.1.2 Strategies

Having decided to consider abnormal loading effects it remains to decide upon strategy. A decision (D3) must be made whether to attempt to control the loading or, in some manner, to accommodate the loading or both. The frequency and severity of the relevant abnormal loading can be controlled in one or more of the following ways:

i) by eliminating the cause, e.g., by specifying that the use of a gas service system within the building be avoided, by the use of crash barriers to avoid vehicle impact, etc. Zero risk, like perfect safety, is not practically attainable and it may not be possible to eliminate completely the cause of the loading. All that is realistically required is that the probability of severe structural damage due to abnormal loading and hence the risk level be reduced below some specified datum.

ii) by reducing the effect of the abnormal loading e.g., by the provision of venting for an explosion, by the use of shock absorbers for vehicular collison, etc. In this manner the resulting forces and their effects due to abnormal events may be reduced to a satisfactory level e.g., below that of normal forms of loading.

iii) by protecting the structure of the building, for example by enclosing any natural gas or steam or fuel lines within specially designed distribution ducts. These services are then isolated and any incident would be prevented from affecting the building's structure. In planning the layout of the building, particularly the ground floor and parking areas, the intelligent designer can do much to lessen the probability of occurrence as well as the risk associated with an abnormal incident, especially vehicular collision. The building can also be protected in other ways; for example the probability of a bomb explosion can be reduced by the use of security personnel or selective tenancy. Conversely various buildings may be more prone to certain forms of abnormal loading as a consequence of the tenant or their use. Clearly the designer should be aware of the social as well as the technical implications of design decisions.

The role and authority of the regulatory agency in attempting to control the loading is very different from that of the individual designer. The regulatory agency is able to influence and alter building practices whereas the designer usually operates within the context of existing practice. An example of this authority is the situation in France with regard to the use and location of gas service systems (see Chapter 8). It must be emphasized that in seeking to control the loading both the regulatory agency and the designer are attempting to ensure that the satisfaction of the performance equation is unnecessary and that the structure need not be designed to accommodate abnormal loading.

If the decision (D3) is made to accommodate each significant abnormal loading or a single representative abnormal loading, the regulatory agency must then decide (D4) whether to do so directly or indirectly. If it opts for the latter the agency must decide (D5) upon implicit or explicit regulation. For the designer these decisions relate to the nature of compliance. Whatever the choice (2, 3 or 4 in the flow charts) it follows that the behavioral response of the structure must be affected by the provisions to accommodate abnormal loading.

The regulatory authority could decide that indirect accommodation without explicit structural design criteria is adequate provision against progressive collapse. This choice would be made after having carefully evaluated the abnormal loading problem. In some circumstances, for instance, in certain earthquake zones this could be a responsible and valid approach. Presumably in a code of practice some comments regarding the benefits of continuous ties and ductility and attention to anchorage details would constitute good practice advocacy and would imply that, if and when an abnormal loading should occur, the structure will behave satisfactorily.

Alternatively it could be decided (D5) that while an indirect approach to accommodating or resisting abnormal loading is acceptable, some explicit structural design criteria are necessary. These, in order of priority, are likely to specify that:

i) certain minimum tie forces are to be resisted at various horizontal and vertical locations within the building,

ii) continuity of resistance is to be preserved within and between certain important components of the structure,

iii) returns be used on isolated vertical walls and that vertical cores be distributed throughout the building,

iv) the choice of both the type and location of walls has some bearing on the nature and magnitude of an explosion and certain materials and floor layouts are to be preferred.

The main feature of this latter approach is that the behavior of the structure and its component parts are adjusted in an essentially empirical manner to accommodate some form of abnormal loading that does not necessarily have to be quantified. The fact that it is difficult to define and quantify any of the abnormal loadings is one reason for adopting this sort of approach to cope with the abnormal loading problem. There will be instances where, for certain sub-classes of building or particular buildings, it may be desirable or necessary (i.e., unavoidable) to design the structural system and possibly some or all of its components to be able to withstand specific abnormal loadings. In other words the performance equation

$$R^B \geq R^P$$

may have to be satisfied for the structural system and for those components where it is deemed necessary. In these circumstances the permissible performance conditions (i.e., R^P) and the acceptable behavioral assumptions (i.e., R^B) must be explicitly and comprehensively specified. Moreover both sides of the performance equation should reflect the economic and behavioral realities of the situation. For example large deformations are involved, structural damage does occur and, subsequent to an abnormal event, the building may only have to be safe for a period long enough to permit evacuation or repair. Since conditions, procedures and concepts are involved that are not customarily used in structural design there will, not unnaturally, be some reluctance on the part of the regulatory agency as well as the designer to adopt these approaches. Whatever is specified here is important since it is likely to have considerable impact on the analysis and configuration of buildings and on the likelihood and direction of future research and development work.

There are essentially two different but related approaches to strategy 4 in Figures 10.1 and 10.2. In the one approach the structural integrity of the overall system is assured by designing all or critical components to withstand the actual application of the abnormal loading, thus limiting damage and localizing loading effects. The other approach must be used when the extent or nature of the damage due to the abnormal loading is such that structural components are caused to fail. Damage may be extensive but the system can be designed to "bridge over" or somehow accommodate this damage by the development of an alternative path of resistance for the forces formerly resisted by the failed components; hence the concept of notional removal of building elements, i.e., a damaged system approach. In either approach a realistic behavioral model should be used to evaluate structural response. Failure theories should incorporate post-elastic material response, large deformations and, where necessary, catenary action in horizontal components.

It should be appreciated that in any building it may be necessary to utilize both approaches. For example, the flank walls in residential buildings are often critical for purposes of overall stability and may have to be designed to resist the relevant abnormal loading. It may nevertheless not be economically feasible to design internal walls in a similar manner. To ensure system stability the damaged system approach may be adopted.

The flow chart in Figure 10.3 both summarizes and demonstrates the nature of the problem of designing a building to accommodate directly

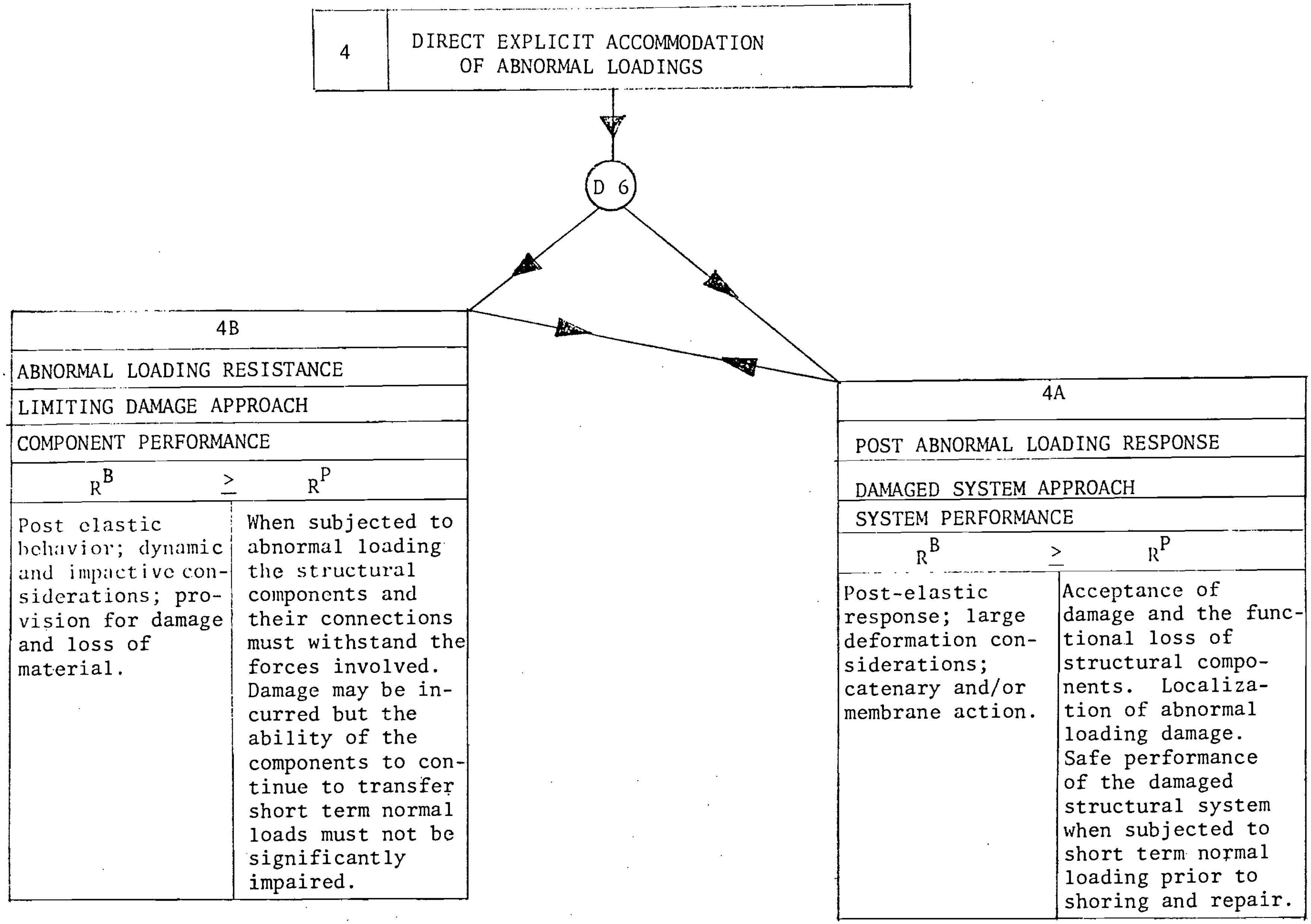


Figure 10.3 - Extension to the Regulatory Flow Chart (Figure 10.1)

a specified abnormal loading. This is merely an extension of Figure 10.1.

In concluding this outline of the regulatory problem it must be emphasized that none of the strategies (i.e., 1, 2, 3 or 4 in Figure 10.1) is mutually exclusive. The linkage illustrated in Figures 10.1 and 10.3 clearly shows that once the decision is made to consider abnormal loadings (D2), the regulations must, in some manner, provide for the possibility that loading control (1), indirect but explicit accommodation (3), and direct explicit design (4) may all have to be specified. Moreover in attempting to cover all sub-classes of building within one set of regulations all four alternatives may have to be utilized.

10.2 Comparative Evaluation and Synthesis

The purpose and content of structural regulations dealing with the related problems of avoiding progressive collapse and accommodating abnormal loadings differ from country to country. Nonetheless the various regulatory responses to the problem do all fit into the general framework of the flow chart shown in Figure 10.1. Following this format the regulations in the countries previously considered are categorized and summarized in Tables 10.2 to 10.12. These tables emphasize the degree of comprehensiveness of the regulations and the various strategies adopted. More specific details are found in the earlier chapters.

These fairly recent regulations do demonstrate some recognition of the fact that abnormal (or localized, exceptional, accidental, extreme, etc.) forms of loading do occur and can have structural consequences. This acknowledgment has very important legal and professional implications, particularly if it is stated explicitly. Even if no other information or guidance is provided one must infer that the regulatory agency is presuming the problem to be tractable and the designer to be able and willing to resolve it. It is highly significant that countries such as Sweden and the United Kingdom, which have collected information on incidence and consequence and have studied the risk associated with abnormal loadings, have formulated the most stringent and comprehensive regulations and the most extensive design guidelines.

TABLE 10.2

Country	United Kingdom
Relevant Building Regulation	CP110 (1972)
Sub-Classes of Building or Type of Construction	Structural concrete buildings
Types of Abnormal Loading	Sufficiently general to incorporate all forms of abnormal loading
Regulatory (or Design) Strategies	
1 Loading Control	Recommended approach for vehicular impact
2 Indirect Implicit Accommodation	
3 Indirect Explicit Accommodation	To generally ensure stability irrespective of type of loading, minimum horizontal (in all buildings) and vertical (in buildings with 5 or more stories) tie forces are specified. In addition a minimum ultimate lateral loading (≥ 1.5% of the dead loading) is specified.
4A Direct Explicit Accommodation - System	Additional requirement applicable only to buildings of 5 or more storeys that are tied horizontally but not vertically and usually involving precast elements--in damaged system assume the extent of damage is the removal of one vertical component. Large deformations are permitted.
4B Direct Explicit Accommodation - Component	Additional requirement applicable only to buildings of 5 or more storeys that are tied horizontally but not vertically and usually involving precast elements--assume 5 psi ($34 kN/m^2$) as a representative static loading based on a gas service system explosion.

Table 10.3

	Country	Sweden
	Relevant Building Regulation	SBN 22:35 (1973) SBN 67 (1967)
	Sub-Classes of Building or Type of Construction	All buildings but load bearing wall structures in particular (≤ 16 stories).
	Types of Abnormal Loading	Most of the relevant forms of abnormal loading are specifically mentioned.
Regulatory (or Design) Strategies	1 Loading Control	This type of approach is mentioned as an alternative to the $\boxed{3}$ and $\boxed{4A}$ combination.
	2 Indirect Implicit Accommodation	
	3 Indirect Explicit Accommodation	To ensure integrity of structural components and their joints, irrespective of the abnormal loading type, minimum horizontal and vertical tie forces are specified.
	4A Direct Explicit Accommodation - System	To ensure integrity of the structural system use alternative path approach. Extent of damage in damaged structure -- a cubic volume of not less than (i) story height + floor depth, or (ii) H/10, or (iii) L/20. Large deformations are permitted and utilization of catenary action is recommended.
	4B Direct Explicit Accommodation - Component	In the event that a particular abnormal loading is identified and defined, then critical components and their joints could be designed to withstand this loading.

Table 10.4

Country	Denmark
Relevant Building Regulation	Bygnings reglement (1972)
Sub-Classes of Building or Type of Construction	All buildings of more than 6 stories.
Types of Abnormal Loading	Not specified - general coverage.
Regulatory (or Design) Strategies	
1 Loading Control	
2 Indirect Implicit Accommodation	
3 Indirect Explicit Accommodation	Irrespective of abnormal loading, a minimum tie force is specified in both the vertical and horizontal directions for all exterior wall and floor or roof components.
4A Direct Explicit Accommodation - System	System damaged to the extent that any wall and floor or roof components (panel or, slab) in any room that is adjacent to the exterior can be rendered non-functional. Abnormal loading not specified. Little guidance provided on behavioral aspects of this alternative path approach.
4B Direct Explicit Accommodation - Component	

Table 10.5

	Country	West Germany
	Relevant Building Regulation	DIN 1045 (1972) [DIN 1055 (1971)]
	Sub-Classes of Building or Type of Construction	Structural concrete buildings in general; precast systems in particular.
	Types of Abnormal Loading	With the exception of vehicular impact no explicit reference is made to any abnormal loading.
Regulatory (or Design) Strategies	1 Loading Control	
	2 Indirect Implicit Accommodation	
	3 Indirect Explicit Accommodation	General Loading: For buildings with precast elements and more than 72'0" in height, minimum tie forces are specified in both vertical and horizontal directions.
	4A Direct Explicit Accommodation - System	Vehicular Impact: If the functional removal of any vertical element does not impair system stability then the element does not have to be designed as per 4B.
	4B Direct Explicit Accommodation - Component	Vehicular Impact: Vertical load bearing elements are to be designed to accomodate an equivalent static force.

TABLE 10.6

Country	The Netherlands
Relevant Building Regulation	NEN 3850 (1972)
Sub-Classes of Building or Type of Construction	Building structures in general
Types of Abnormal Loading	General reference to localized loading - fire, explosion, collision, etc.
Regulatory (or Design) Strategies	
1 Loading Control	
2 Indirect Implicit Accommodation	Unless a special abnormal loading is identified and then quantified, it is presumed that normal good practice will ensure structural integrity.
3 Indirect Explicit Accommodation	
4A Direct Explicit Accommodation - System	
4B Direct Explicit Accommodation - Component	While neither load nor response are specified within the code design for abnormal loads is required. Information on <u>gas explosion</u> and <u>vehicular collision</u> are available and the designer is presumed to be competent enough to ensure adequate performance of structural components and hence the system.

TABLE 10.7

	Country	Canada
	Relevant Building Regulation	NBC (1975)
	Sub-Classes of Building or Type of Construction	All forms of building construction, i.e., general
	Types of Abnormal Loading	All forms of abnormal loading, i.e., general
Regulatory (or Design) Strategies	1 Loading Control	
	2 Indirect Implicit Accommodation	Abnormal loadings are not in any way specified or quantified; to avoid progressive collapse and ensure stability "good engineering practice" is advocated. No explicit directives are provided but useful guidance and information is given in a Commentary.
	3 Indirect Explicit Accommodation	
	4A Direct Explicit Accommodation - System	By default, the commentary to the code recommends that British practice to be followed.
	4B Direct Explicit Accommodation - Component	

Table 10.8

Country	France
Relevant Building Regulation	Journal officiel: No. 69-88 Construction No. 1299 Installations de Gaz. No. 67-216 Securite contre L'incende
Sub-Classes of Building or Type of Construction	Buildings in general but with particular reference to residential building.
Types of Abnormal Loading	See below.
Regulatory (or Design) Strategies: 1 Loading Control	<u>Gas Service System Explosion:</u> Probability of occurrence and therefore risk reduced by regulation of gas usage and distribution, independent of structural regulation.
2 Indirect Implicit Accommodation	Presumably normal good practice is satisfactory once the risk or gas explosion has been reduced.
3 Indirect Explicit Accommodation	
4A Direct Explicit Accommodation - System	
4B Direct Explicit Accommodation - Component	

Table 10.9

Country	C.E.B.
Relevant Building Regulation	International recommendations for the design and construction of large panel structures (1967)
Sub-Classes of Building or Type of Construction	Large panel buildings
Types of Abnormal Loading	Not Specified Apart From Exceptional Errors of Execution
Regulatory (or Design) Strategies	
1 Loading Control	
2 Indirect Implicit Accommodation	
3 Indirect Explicit Accommodation	To avoid a "House of Cards" effect both peripheral and internal Horizontal tie requirements in both walls and floors are specified. A nominal lateral load requirement is specified and various general provisions are also relevant.
4A Direct Explicit Accommodation - System	
4B Direct Explicit Accommodation - Component	

TABLE 10.10

Country	Poland
Relevant Building Regulation	The design and construction of buildings made from large panel prefabricated elements. 1975.
Sub-Classes of Building or Type of Construction	Large panel, precast concrete systems
Types of Abnormal Loading	Gas explosion, vehicular collision, etc.,
Regulatory (or Design) Strategies	
1 Loading Control	
2 Indirect Implicit Accommodation	
3 Indirect Explicit Accommodation	Both vertical and horizontal tie forces are specified; tie beams being emphasized. Specific provision is made for wall to floor connections
4A Direct Explicit Accommodation - System	Mention is made of the possible removal of a member, in particular a flank or wall or roof element, and the need to develop both cantiliver and/or catenary action. The approach adopted in $\boxed{3}$ above is assumed to ensure this response.
4B Direct Explicit Accommodation - Component	

TABLE 10.11

	Country	Czechoslovakia
	Relevant Building Regulation	"Recommendations on the structural design of panel buildings" VUPS 1970
	Sub-Classes of Building or Type of Construction	Large panel, precast concrete systems
	Types of Abnormal Loading	Abnormal (extraordinary) loads in general; impact and explosion identified.
Regulatory (or Design) Strategies	1 Loading Control	
	2 Indirect Implicit Accommodation	
	3 Indirect Explicit Accommodation	Horizontal tie forces are specified. Vertical tie forces are recommended. Tie beams are emphasized.
	4A Direct Explicit Accommodation - System	
	4B Direct Explicit Accommodation - Component	Provided the requirements of $\boxed{3}$ are satisfied it is not necessary to calculate the effect of an abnormal loading.

Table 10.12

	Authority	Comecon
	Relevant Building Regulation	Draft design recommendations for large panel buildings in Socialistic countries (Moscow) Feb. 1970
	Sub-Classes of Building or Type of Construction	Large panel, precast concrete systems
	Types of Abnormal Loading	Accidental loads in general but gas or other explosions in particular
Regulatory (or Design) Strategies	1 Loading Control	
	2 Indirect Implicit Accommodation	
	3 Indirect Explicit Accommodation	
	4A Direct Explicit Accommodation - System	Alternative path approach for notional removal of one wall panel recommended. Design based on service level gravity dead and gravity live loads at limiting material strengths. Large deformations (cracking, inelasticity) may be considered. Continuity, deformability and layout considerations are advocated.
	4B Direct Explicit Accommodation - Component	Permissible but not mandatory except where the alternative path approach is inadequate. Actual design loadings are not specified in the regulation.

COUNTRY	TYPE OF REGULATION CONCERNING OVERLOADING		
	A. Functional requirements intended to reduce the risk associated with progressive collapse due to local damage below an acceptable level	B. Strength requirements (design for special forces) Design for loss of carrying capacity in specific parts of the building (alternative path approach).	C. Rules of thumb concerning the strength and layout of structural connections.
Nordic Conc. Assoc.	■ 1970		■ 1970
Sweden	■ 1973		[V] ■ a) 1973
Denmark		■ 1969	■ 1969
United Kingdom		■ 1970	[L] '70 [F] '71 [B] '72
Canada	■ 1970	■ b.)	[L] b.)
U. S. A.	■ 1972	■ b.)	[L] b.)
France (CEB/CIB)	[L] 1969		[L] 1969
Holland			[L] 1970
Italy			[L] 1969
Poland			[L] 1969
Czechoslovakia			[L] 1970

■ Applicable to all buildings

[L] Applicable only to large panel structures

[F] Applicable only to framed structures

[V] Applicable to all bearing wall structures

[B] Applicable to concrete buildings

a) the commentary to the regulations envisages certain exceptional circumstances

b) In the absence of specific regulations the relevant British recommendations are adopted.

Table 10.13 Comparative Summary of Abnormal Loading Regulations (See Ref.[1.12]By Granstrom and Carlsson).

While the various regulations may acknowledge the incidence of abnormal forms of loading and the related possibility of progressive collapse there is little consistency in their treatment of the problem. The nature and relative significance of the problem within each country either differs or is seen to differ and, as a consequence, it is difficult and perhaps not even valid to compare regulations. Tables 10.2 to 10.12 provide a fast and efficient means of comparing the various regulations. However, oversimplification can lead to distortion or error as is evident in Table 10.13, taken from Granstrom and Carlson's report [1.12], where the regulatory situation in the U.S. is clearly misrepresented. Table 10.13 is instructive in that although most regulations have been revised since 1973, it does provide some historical perspective. Accordingly an attempt will be made to evaluate the various regulations with regard to a number of common criteria.

10.2.1 Scope

The CIB Commission W23A made the point that the problem of progressive collapse is of a general nature and should be taken into account in all types of buildings regardless of material, type of structure and construction methods used [1.11]. The various regulations do seem to differ on this point. Consider the following:

i) Material of Construction:

While the probability of an abnormal loading occurring may be independent of the construction material, this is not true of the vulnerability of a building to abnormal loading and its propensity to collapse progressively. In other words structural response may be greatly dependent upon construction material. Any truly comprehensive study of building regulations must therefore include both clay and concrete masonry construction. Furthermore the avoidance of progressive collapse during construction is of particular significance for both structural steel and timber buildings. The generality of code clauses is closely related to the overall regulatory system that exists in each country. In those countries where code criteria involving loading and performance are independent of material, e.g., Sweden and Canada, requirements for the accommodation of abnormal loading and the avoidance of progressive collapse apply to all buildings. Where the regulations are material dependent, as in the United Kingdom, there is no generality of application and as a result buildings constructed in different materials are likely to have different levels of risk against abnormal loading. There is also the possibility that in being progressive and professionally responsible a sector of the building industry could economically and otherwise penalize itself. Therefore in order to avoid this type of economic inconsistency any regulatory action should be as general as possible. It is perhaps for this reason that regulatory action in some countries, e.g., Denmark and the U.K., applies only to buildings over a certain height. It is not coincidental that the low-rise or walk-up multi-unit residential building is unaffected by most code provisions for abnormal loadings or progressive collapse.

ii) Type of Construction:

The level of risk associated with any abnormal loading will depend upon the nature and quality of both construction and engineering input (if any) as well as building use and occupancy. There is some degree of consistency in that most regulations do recognize the potential vulnerability of bearing wall, particularly precast concrete, construction. While large panel bearing wall systems may be regulated, no specific mention is made of the propensity of structural systems with cast-in-place flat slab floors to collapse progressively and their vulnerability to debris and impact loads. The safety of unbonded post-tensioned floor systems is especially pertinent in North America [10.1].

Many regulations do recognize that the risk level may vary with building height. While the maximum height chosen to identify low rise buildings may vary between 30.0' (4 stories) and 72.0' there does appear to be a consistent attempt to exclude low rise buildings from regulation. Precautions are however usually taken with medium and high-rise buildings. This may owe as much to political and economic considerations as to logic; for example, a 3 or 4 story walk-up type residential building is probably much more vulnerable to progressive collapse than a multi-story commercial building.

To define satisfactorily the scope of any regulations requires a much better understanding of relative risk and benefit than is presently possible. The inadequacy of data coupled with a general lack of experience precludes completeness or consistency in this regard.

10.2.2 Intent

The two principal issues that should be but are not generally spelled out, are:

i) that the avoidance of progressive collapse is a general design requirement irrespective of the "normality" or "abnormality" of the loading. Normal loadings can have localized effects and conversely abnormal loadings are not the only forms of localized or for that matter accidental or extreme loads.

ii) that abnormal loadings may have to be accommodated in the overall sense that structural performance be safe and, if necessary, serviceable.

The British and Swedish regulations are reasonably unambiguous in this regard but several of the codes do tend to blur these issues. A related consideration is that the intent of any regulation, whether explicitly stated or not, must make some provision for the realities of existing practice. The Canadian code is an example of this dichotomy between regulatory intent and its practical resolution.

10.2.3 Loading Control

One advantage of at least identifying various forms of abnormal loadings is that the control of this loading then becomes an explicit design option. This is of course a very important consideration from at least three viewpoints:

i) the structural designer is made aware of the problem and then has the option of reducing risk either by elimination of the loading, or by reduction of the forces involved or by protection of the structure in some manner.

ii) the regulatory agency responsible for structural considerations can ensure that the designer is made aware of and given responsibility for resolving the problem. In addition this agency can identify high risk situations and make suggestions as to how the risk can be reduced.

iii) regulatory authorities or planning agencies not directly concerned with structural considerations, e.g., gas distribution authorities, security or civil defence agencies, etc., can act or be caused to respond. In France, for example, the structural risk associated with the use of gas in building was reduced by means of regulations for the use and distribution of gas.

One important consideration is that of venting. It is common industrial building practice to reduce the effects of an explosion by means of deliberate venting. This may be more difficult to accomplish in residential or commercial buildings. While work has been done on the venting capabilities of glass windows, little is known of the venting capabilities of typical lightweight partitions, non-load bearing walls, etc. Whether the intent is to control or to accommodate an explosive loading it is necessary for reliable data on the venting characteristics of typical building components to be developed.

10.2.4 Indirect Accommodation

Irrespective of whether the provisions are implicit or explicit this approach requires neither mention nor consideration of either loading or performance criterion. The presumption is that by following certain rules or practices the probability of structural failure is reduced to an acceptable but unspecified level.

This study has not sought to evaluate the meaning of "good normal practice" in each country. Even if this were possible "good practice" is a subjective criterion and in most countries it is apparently recognized that normal practice may not be good enough to sustain abnormal loads or avoid progressive collapse. This is particularly true of novel or different structural systems, construction methods and building materials. A much better argument could be made in the case of buildings in severe earthquake zones where normal practice could be considered to be adequate practice with regard to most abnormal forms of loading. None of the codes studied makes this argument presumably because, with the exception of Canada, seismic action is not ordinarily a significant consideration.

Given that the indirect accommodation strategy is to be used (either as an option or the only design procedure), it remains to evaluate and compare those criteria that have been explicitly stated in the various regulations. Certain provisions such as those that relate to building layout, continuity, anchorage and detailing are important and of general value. A number of these regulations or, more often, their commentaries devote considerable space to these issues. In each chapter, especially the British, Swedish, Danish and Canadian chapters, an attempt is made to emphasize and illustrate many of these aspects of what could be called "good abnormal loading/anti-progressive collapse practice". Of particular concern however are the tie (force or steel area) provisions of the various codes.

10.2.5 Tie Considerations

Table 10.14 summarizes tie requirements either as forces or areas of reinforcement, insofar as they can be categorized. It is evident that the force 20 kN/m (1370 pounds per foot) is the only tie force requirement with any degree of universality (see the British, Swedish and Danish provisions). This value may be related to the proposed amendment to the CEB recommendations [8.9] where a service load tie force of 8 kN/m is suggested. For a limiting stress value this tie force would be equivalent to 20 kN/m, i.e., for a working stress level of 0.4 f_y. For f_y = 50 ksi the equivalent area of steel required for a 1370 pound per foot tie force is 0.027 ins^2/foot (or 0.58 cm^2 per metre). For L = 6m (19.7 feet) this would result in 3.48 cm^2 or .162 ins^2 of steel reinforcement which, in turn, is comparable to the tie beam requirements in the Polish regulations. The West German requirements are not necessarily less stringent than a 20 kN/m requirement and this value could be presumed to be a consensus lower bound for the magnitude of the tie force.

Only the British and Swedish regulations attempt to provide

TABLE 10.14: Summary of specified tie requirements.
See notation on following page.

TIE FORCE \ COUNTRY AND CODE		UNITED KINGDOM CP110 (1972)	SWEDEN SBN 22:35 (1973)	DENMARK (1972)	WEST GERMANY DIN 1045 (1972)	C.E.B. Bull #60 (1967)	POLAND (1975)	CZECHOSLOVAKIA (1970)
Vertical Direction		For $n < 5$ $F = 0$ For $n \geq 5$ $F \geq \frac{0.4\, bdf_y}{100}$ (F in kN)	$n \ngtr 4, F = 0$; $n \ngtr 8, F = 20$ in ext. elements only; $n \ngtr 16$, $F = 20$ in all elements (kN/m units)	Every normal cross-section in the exterior wall or in the floor or roof, $F = 20$ kN/m.	Vertical continuity not specified but see wall to floor tie requirements (below).	Not specified	At roof level: $A_s \geq 1\ cm^2/m$ of wall i.e. roof to wall tie.	Not specified but generally recommended for a vertical tie force of at least the gravity load of the wall panel involved.
HORIZONTAL DIRECTION	Peripheral (perimeter, external)	$F \geq (20 + 4n)$ But $\ngtr 60$ kN	For $n \ngtr 4$, $F = 20$ kN/m, but $\nless (W_D + .33W_L)L$ in main load bearing direction, for $4 < n \leq 16$, $F = 20$ kN/m, but in main load bearing direction $F = 20[1 + .1(n-4)]$ but $\nless (W_D + .33W_L)\,L\,[1 + .1(n-4)]$ kN/m		Tie beam (note 1500 m^2 area limit) force of 30 kN	In the English translation [8.9] the intent or meaning of the relevant clause is not precise but the only explicit quantitative requirements are: $F \geq 5$ kN per metre of wall or $\Sigma A_s \geq 2\ cm.^2$ where the steel may be distributed over the height of the wall or in the tie beam.	Horizontal tie beams for: (i) floor slabs supported on 3 or 4 edges or on two edges if $L \leq 4.80$ m. $\Sigma A_s \geq 2.3\ cm^2$ (ii) floor slabs supported on two edges with $L \leq 6$ m. $\Sigma A_s \geq 3.4\ cm^2$ (iii) $\Sigma A_s \geq .8L\ cm^2$ but $\geq 2.3\ cm^2$	Horizontal tie beams: (i) $F \geq 15L$ kN where L is the distance between bearing walls (ii) in tie beams connecting exterior bearing walls the tie beam parallel to the span is to designed for $F \geq 15$ kN per metre of connected wall width.
	Internal	$F \geq (20 + 4n) \times \frac{(W_D + W_L)}{7.5} \frac{L}{5}$ kN/m But $\nless 60$ kN/m			Tie force of 15 kN (usually located in joint between and parallel to floor panels).			
	Columns or Walls	In external elements only (per metre for walls) $F \geq 2(20 + 4n)$ But $\ngtr 120$ kN and $\ngtr (h/2.5)(20+4n)$ $\nless 3\%\ W_v$			Floor to wall: tie at top and bottom for external walls; at top only for internal walls; $F = 7$kN per metre of wall.			
Steel Stress Limit		f_y	f_y	f_y	f_y	f_{sw}	—	f_{sw}

Notation to Table 10.14

n	number of stories or floors
b	width of element (section) considered
d	effective depth of the section concerned
W_D	dead loads
W_L	live loads
W_V	total vertical load at the level concerned
F	tie force value
A_s	cross-sectional area of reinforcement
ΣA_s	total area of tie reinforcement (usually in the tie beam)
f_{sw}	the allowable steel stress at service or working load
f_y	the yield stress of the steel

for the fact that risk will vary with building height and both these requirements are illustrated in Figure 10.4. Clearly for a building of approximately 16 floors or less the British and Swedish tie requirements are comparable. In addition their tie provisions are much in excess of those specified by the other regulations considered (see Table 10.14).

Further comments that arise from a comparison of tie force requirements are:

i) in general tie provisions in the vertical direction leave something to be desired. For example only the Polish and Danish regulations focus on the need for a roof to wall connection. All of the regulations either ignore or avoid vertical tie considerations in building of less than 5 or 6 stories. The Czech regulations do make the good point that as a lower bound the amount of vertical reinforcement should at least be able to sustain the self weight of the wall element involved.

ii) it would seem to be preferable to specify tie requirements in terms of forces rather than amounts of steel. This does permit flexibility of bar placement and fosters a better understanding of the issues involved. Most regulations adopt this approach and relate these forces to yield rather than to service level steel stresses.

iii) even allowing for the fact that the translation may be at fault the precise intent of some of the tie requirements is sometimes ambiguous. The use of terms such as peripheral, internal, and longitudinal in conjunction with vertical and horizontal elements can give rise to some confusion. Because different forms of construction are involved it is not always clear where the tie reinforcement is to be placed. For example does the term "tie beam" refer only to the junction of the floor and walls or are vertical tie beams (e.g., at the junction of wall panels) also implied? Another regulatory item that often seems in need of clarification and comment is the precise location of the interior tie reinforcement when precast floor elements are used. In many instances a few illustrated examples would be of interpretative assistance.

iv) the West German requirements do stress the fact that the smaller the precast floor unit, the more likely it is to be supported at two edges only. A minimum width of 2m (6'-7") is quite restrictive but it is

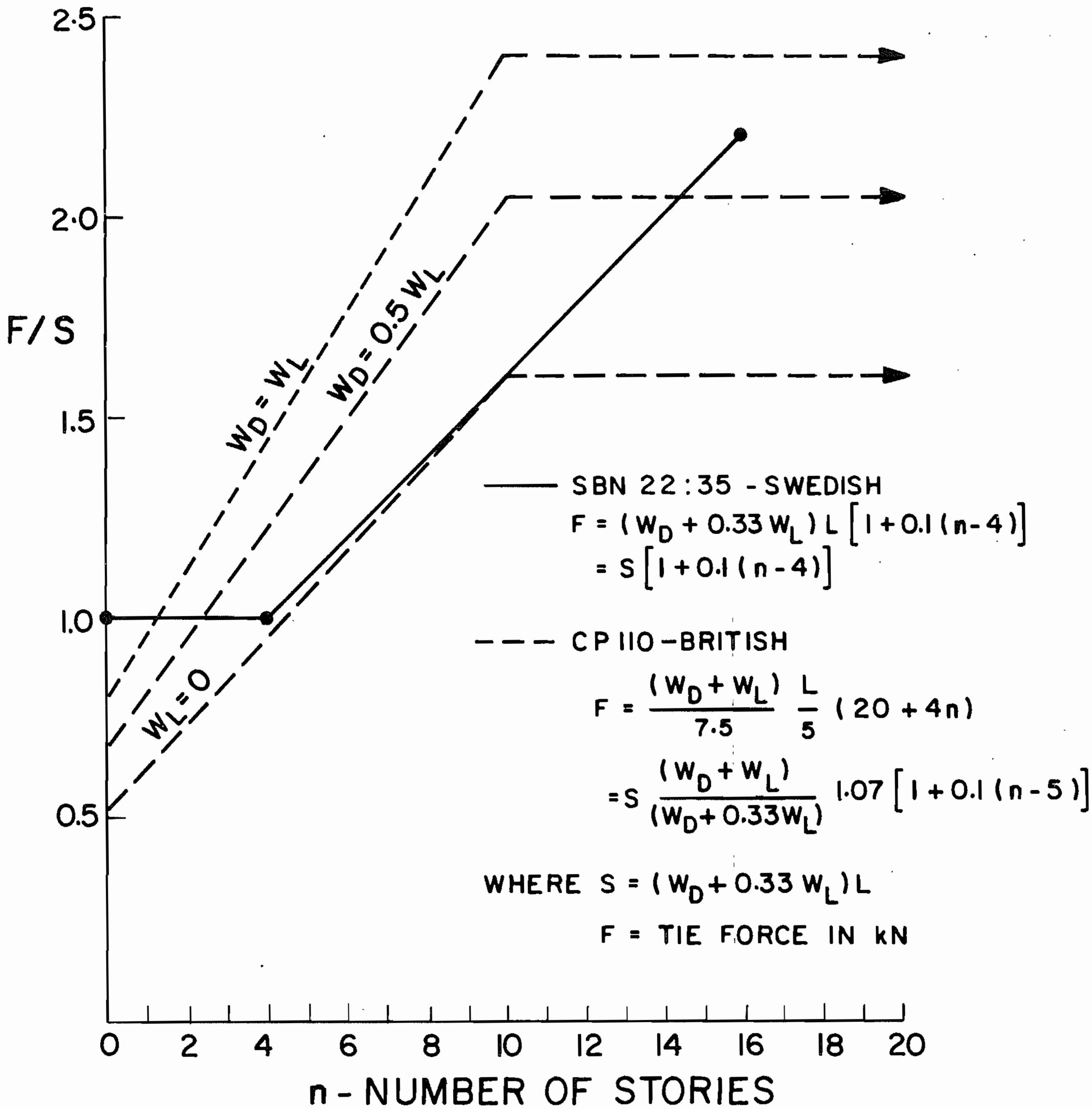


Fig. 10.4 HORIZONTAL TIE FORCE REQUIREMENTS [3.5]

important that the benefits of three or more edges of support be recognized. This concern is also reflected in the Polish regulations but the emphasis is not directly placed on the width of the floor element.

v) one approach which can, but does not necessarily, improve the abnormal loading/progressive collapse capabilities of the structure, is the specification that a minimum lateral force be applied to the building. This is usually some proportion of the vertical loading at the level concerned and is, in effect, an indirect but explicit means of ensuring general structural stability under lateral loads. This provision does not necessarily affect tie requirements in either the vertical or horizontal direction. Various percentages of the vertical dead load are specified e.g., in the U.K. 1 1/2% of the total dead load at the level concerned.* A similar approach but applied to individual components rather than to the building as a whole, does have a better chance of influencing tie reinforcement. For example the CEB recommendations require that all members be designed to accommodate a horizontal force of 1% of the load on the member. Section 19.8.3 of DIN 1045 is rather more explicit and requires that the tie be capable of resisting a tension force of 1% of the vertical load (dead plus live) on the element involved. Similarly for column or wall ties in the horizontal direction CP 110, Section 3.1.2.2.c., requires 3% of the total ultimate vertical load to be taken by the ties. This latter approach does at least ensure that tie reinforcement is provided.

In discussing the relative merits of the tie reinforcement provisions of the various regulations one should not lose sight of the principal issues involved. A crucial decision is whether or not to specify the reinforcement. Outside of moderate to severe earthquake zones such specification is tacit admission of the fact that abnormal loadings need to be accommodated. Consequently this reinforcement, in addition to any normal loading function, must be able to contend with explosive and impactive effects. Accordingly the quantity and the detailing (i.e., considerations of distribution, location, continuity, anchorage, etc.) of this tie reinforcement is important. Given that ties are required it is evident that the amounts of steel involved in the various regulations are not particularly

* This is a minimum "ultimate" value that is roughly equivalent to 1% at service levels of loading.

onerous. What is significant is that it is necessary and that it has to be correctly detailed. Since this reinforcement must either resist the abnormal loading or contend with its after-effects its detailing will not necessarily be the same as existing 'good' practice. It is only necessary to refer to the recent paper by Rhodes [2.23] to appreciate that, in a number of respects, existing North American detailing practice is inadequate against explosion. In the U.K. where the most experience in designing against abnormal loading and progressive collapse has been accumulated, it was found to be necessary to produce a special text [2.22] to elaborate on the detailing aspects of CP 110. Reference should be made to Chapter 2 for further comment on this aspect of both the regulatory and the design problem.

10.2.6 Direct Accommodation

Three sets of regulations namely the British CP 110, the Swedish and the Comecon recommendations, advocate the design strategies that are generalized and summarized in Figure 10.3. In each the interrelationship between the "damaged system approach", (i.e., 4A or the alternative path method) and the "limiting damage approach", (i.e., 4B) is acknowledged. Since most buildings will have one or more critical components whose failure will always have disastrous consequences (e.g., flank walls) it is likely that these approaches will usually have to be used in combination. For instance, any potentially critical component will have to be designed to withstand the abnormal loading in order that, for system integrity, an alternative path/damaged system approach can be used to ensure acceptable post-abnormal loading response.

It is somewhat paradoxical that the West German regulations adopt the design approach outlined in Figure 10.3 to accommodate vehicular impact. However when it comes to abnormal loadings in general and their effects on precast concrete buildings in particular, the West German regulations evidently prefer the indirect approach involving explicit stability criteria.

It is also interesting that although both the Danish and Polish regulations refer to the damaged system/alternative path option they do

Country and Regulation	Load Factor			Stress Factor		Remarks
	Dead W_D	Live W_L	Wind W_W	Concrete	Steel	
U.K. CP 110	1.05	a) ≤ 1.05	a) ≤ 1.05	$.52\ f_{cu}$	f_y	See Section 2.3.3.1. f_{cu}=cube strength
Sweden SBN 22.35 (See [3.4])	1.0	b) 1.0	0.5	c) -	f_y	See Appendix S i.e., Commentary to Chapters 21 & 22 of SBN 67.
Denmark (See [4.2])	1.0	.33	.25	$1.8\ f_{cw}$	$1.8\ f_{sw}$	f_{cw}, f_{sw} are service load stress levels.
Comecon [9.8]	1.0	1.0	0	c) -	f_y	

Table 10.15 - Loads and Stress Levels to be Used in Considering the Performance of the Damaged Structural System

Notes:

a) the load factor values depend on the period of time involved and the probability of occurrence of individual loads or combinations thereof.

b) this load factor is not explicitly stated. Apparently [3.3A] the value specified in the Nordic Concrete Association Report of January 1970 [10.3] is to be followed.

c) apparently the equivalent "yield" or the usual limiting strength is to be used.

10.2.6.2 Limiting Damage Approach

The two principal issues are:

A. the abnormal loading or loadings to be considered, and

B. the behavioral theory to be adopted.

A.1 Explosion:

In CP 110 an equivalent static overpressure of 5 psi (720 psf or 34 kN/m^2) is specified. The draft of the new Swedish regulations similarly specifies 7.5 psi or 3.75 psi depending on the window area or natural venting involved. The Dutch have proposed a rather more comprehensive and rational approach [6.5]. This not only justifies the use of an equivalent static overpressure but also makes provision for venting and suggests that, in most instances, the peak overpressure is probably much less than 5 psi [see Figure 6.1].

All the above loadings are intended to simulate a representative gas service system explosion. How representative this is of explosions in general is debatable [10.2]. Moreover the magnitude and nature of the representative explosive loading has been the subject of some controversy. Given the frequency of gas explosions relative to bomb or other types of explosions [1.1] it is probably more logical to use gas as the datum abnormal explosive loading.

Only a small number of walls in relatively few buildings are ever likely to be designed to individually withstand an explosive loading. It is unlikely that any floor or roof elements need to be designed to directly resist an abnormal loading of this nature. While this may have been implied by earlier British regulations, CP 110 is quite explicit on this point. Given that only a relatively small number of wall components are involved it should also be appreciated that the amount of reinforcement and its placement is not really of much economic consequence. Even for 5 psi the quantities of reinforcement required within and around the vertical components comprising a flank wall are not particularly significant (see Chapter 2).

A detailed review and assessment of explosions and their effects is obviously beyond the scope of this report but two points are worth making:

i) anyone with some familiarity with the readily available North American literature on explosive loadings and their effects will appreciate than an incident involving a peak overpressure of less than 10 psi is categorized as a low pressure explosion and is comparable to the threshold of ear damage [10.4].

ii) in some respects the explosive loading value is a false issue. Its value is of secondary importance in a design or an economic sense but because most building designers have little or no experience of explosive effects and because whatever value is chosen is much greater than, say, normal wind or live load values, there is a tendency for this abnormal loading to generate anxiety and thus receive relatively more attention than it warrants.

A.2 Vehicular Impact:

The new Swedish [3.6] and the German regulations [5.2] both specify vehicle impact loads. Both codes specify various equivalent static horizontal loads of reasonably comparable magnitude. A number of related studies are available [6.6] and there should be no difficulty in assessing these specified loads from a North American standpoint.

Vehicular impact and explosion are the only abnormal loadings that are quantified in the regulations considered. Their significance is confirmed by the findings of loading studies [1.1] but the possibility of debris loading, especially the impact load due to falling structural debris, should not be overlooked. This loading is not quantified in any of the regulations but, if only to establish tie forces, this form of loading deserves more attention.

B. Behavioral Theory:

The behavioral model to be used in analyzing the response of the wall section that is subjected to abnormal loading, depends upon the nature of the load prescribed and the performance desired. None of the regulations considered are particularly explicit on either of these two issues. If however the abnormal loading involved is specified as an equivalent static force then the specification of limiting stresses can automatically prescribe the behavioral model and the requisite performance criterion. CP 110 is the only regulation to expand upon this issue and the stress factors shown in Table 10.15 apply. On applying these values it becomes evident that there is some ambiguity in the use of f_y as the limiting stress value. In designing the wall to withstand the abnormal loading f_y could be used as an upper bound value that ensures elastic response, i.e., the function is in no way impaired. Using the same factors to analyze the alternative path/damaged structure situation it must be presumed that, while f_y may not be exceeded, the steel at certain locations will be yielding in a plastic manner.

Clearly this equivalent static approach oversimplifies a situation which really involves a dynamic forcing function, large deformation and possibly damage. Once the abnormal loading has been withstood it is necessary that the wall continue to function but for a limited time (i.e., until the building is shored or repaired) in a limited manner (under full dead load but with reduced service live and wind loads). While undue sophistication should be avoided it should be borne in mind that in North America there is considerable experience in designing for dynamic loads. For example, the Department of the Army, Navy and the Air Force are jointly responsible for a design manual entitled "Structures to resist the effects of Accidental Explosions" [10.4] and numerous other relevant publications. Many basic texts deal with the nature and structural consequences of both explosive and impactive loads. For the relatively rare occasions when this design approach has to be used there are behavioral as well as educational advantages in specifying the representative dynamic load and the deformation and/or damage limit. For practical

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16. ABSTRACT *(A 200-word or less factual summary of most significant information. If document includes a significant bibliography or literature survey, mention it here.)*

The progressive-collapse related provisions of the building regulations of the United Kingdon, Sweden, Denmark, West Germany, Netherlands, Canada, France, and Eastern Europe are studied in detail. The various regulations are discussed individually for their content, background and interpretation. The report is concluded with a discussion of both building regulatory and design problems associated with the implementation of progressive collapse design requirements. A comparative evaluation is then made of the regulations discussed in the report.

17. KEY WORDS *(six to twelve entries; alphabetical order; capitalize only the first letter of the first key word unless a proper name; separated by semicolons)*

Abnormal loading; building regulations; design process; European; progressive collapse; regulatory process

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