**Zeitschrift:** IABSE reports of the working commissions = Rapports des commissions de travail AIPC = IVBH Berichte der Arbeitskommissionen

**Band:** 36 (1981)

**Rubrik:** Session 1: The influence of forces on the selection of structural form

# SESSION 1

## The influence of forces on the selection of structural form

## Influence des charges sur le choix du système et de la forme des structures

## Einfluss von Lasten auf die Wahl des Systems und der Form von Tragwerken

**Chairman:** Peter F. Adams, Canada

**Co-ordinator:** K. Sriskandan, UK

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I

# Historical developments in the selection of structural form in relation to natural and other forces

Considérations historiques sur le choix du système et de la forme des structures, en relation avec les forces naturelles et les autres forces

Geschichtliche Entwicklung bezüglich der Wahl des Systems und der Form eines Tragwerks im Zusammenhang mit natürlichen und anderen Lasten

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**SUMMARY**
Since man started to build, he has had to contend with gravity, the wind, and often with other forces. The way in which he has selected structural forms to meet his needs has varied with the changing relative importance of different forces, with his understanding of their nature and of structural responses to them, and with the materials and other means at his disposal. Examples of his selections are discussed from before 1800, from the 19th century, and from the 20th century, to illustrate general trends.

**RESUME**
Dès que l'homme se mit à construire, il eut à lutter contre la gravité, le vent et souvent d'autres forces. La manière avec laquelle il choisit la forme des structures devant satisfaire à ses besoins évolua avec le changement d'importance relative des différentes forces, avec la compréhension de leur nature, du comportement de la structure ainsi qu'avec les matériaux et autres moyens à sa disposition. Des exemples sont présentés pour trois périodes, avant 1800, au 19ème siècle et au 20ème siècle et permettent de mieux comprendre cette évolution.

**ZUSAMMENFASSUNG**
Seit der Mensch zu bauen begann, hatte er sich mit der Schwerkraft, dem Wind und oft auch noch mit anderen Kräften auseinanderzusetzen. Das Vorgehen bei der Wahl des Systems und der Form eines Tragwerks, das alle gestellten Anforderungen befriedigt, wurde laufend durch neue Erkenntnisse bezüglich natürlicher und anderer Lasten sowie durch neue Materialien etc. modifiziert. Zur Illustration werden Beispiele aus drei Perioden, von vor 1800, aus dem 19. und 20. Jahr-hundert, gezeigt.

## 1. INTRODUCTION

Natural forms are shaped by the forces acting on them as they grow. We shape the structures that we build. As soon as building starts gravity comes into play. So may other forces like wind, wave, or earthquake. But they do not shape the structure so much as test it; subjecting it to a process of natural selection. We, as designers, propose. Nature, and use, dispose. This has always been so [1,2].

Whatever the primary reasons for building, designers have, of course, always sought to shape their structures so that they will pass the test - so that they will stand in the face of all the forces they will be called upon to bear and will not yield excessively to these forces in any way. Often it has been possible to do this simply by staying within the bounds of earlier choices that had already been shown by experience to be safe. But not always. Any innovation has meant moving outside these limits and has called for some other kind of assurance that all would be well.

Understanding of likely loads and responses to them has then become important. Even today our understanding of both is often less that we should like it to be in relation to the tasks we set ourselves or undertake. We are repeatedly faced with uncertainties about the probable magnitudes of forces, about the dynamic characteristics of some of them or their dependence on some of our design choices, and about important aspects of structural response. A hundred years ago understanding was virtually limited to static loads and statically determinate responses to them. Two hundred years ago a few simple predictions of strengths and determinations of the strengths needed to ensure static equilibrium under gravitational loading were being made almost for the first time. Before that, there was little understanding that was not purely intuitive - and therefore non-quantitative - other than that summed up in the simple laws of the balance.[3,4,5].

In the long prehistory of building, structural forms like simple domical and post-and-beam huts must have been developed by long processes of trial and error which probably differed little from those which taught birds to build their nests. Trial and error still play their part: innovation can still be hazardous. But, as understanding has grown, the hazards have become associated with much bolder steps into the unknown. And - of particular relevance to the topic of the first session of this symposium - they have tended also to be associated with new types of loading or response becoming potentially critical.

To illustrate this, it is possible to consider only a few examples of structures built over a period of some 1500 years. No records survive of the ways in which forms were selected over the major part of this period, at least not in relation to structural criteria of selection. We must therefore use a certain amount of imagination in trying to envisage the bases of selection. But I have chosen examples about which something useful can be said with reasonable confidence. The justifications for what is said will be found elsewhere.

## 2. SELECTION BEFORE 1800

Nothing of great importance happened in 1800. But it does roughly mark significant changes in the choices open to designers, in the requirements

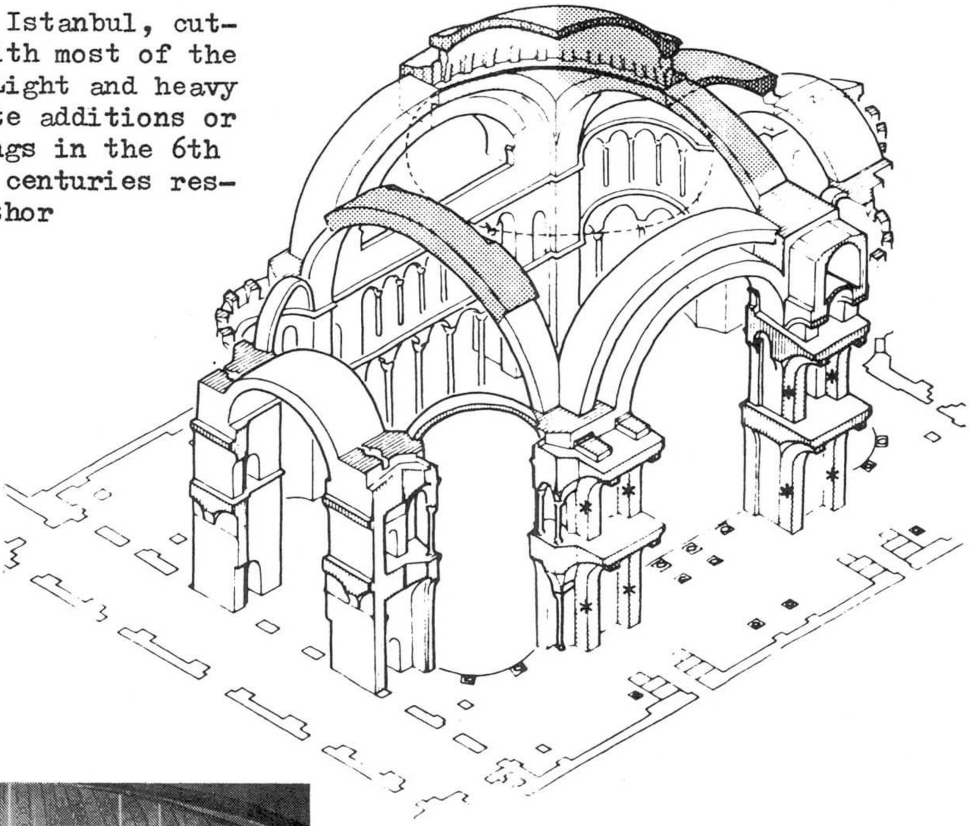

Fig.2 St Sophia, Istanbul, cut-away isometric with most of the vaults removed. Light and heavy stippling indicate additions or partial rebuildings in the 6th and 10th or 14th centuries respectively. © Author

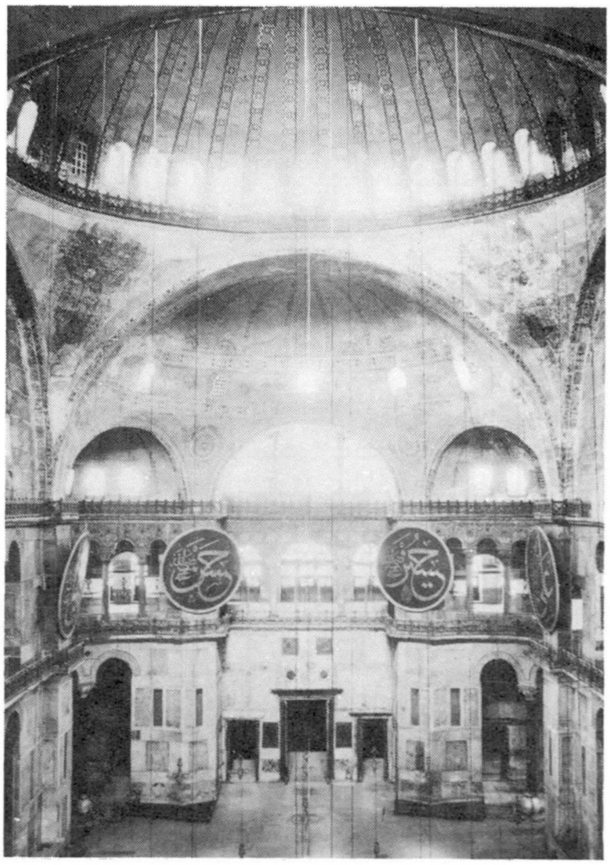

Fig.1 St Sophia, Istanbul looking westward. © Author

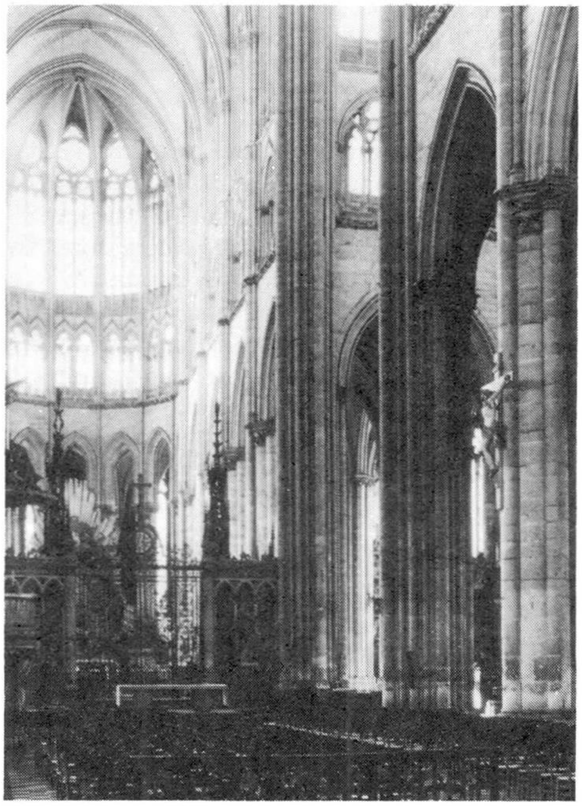

Fig.3 Amiens Cathedral. © Author

they had to meet, and in their structural understanding. Until almost the end of the eighteenth century they had to work almost exclusively with brick, stone, or concrete, and timber. Iron was available only in sufficient quantity for use as cramps or ties. This meant that major structures had to be capable of acting largely in compression, with a possible limited use of timber or iron ties to assist in containing the outward thrusts of arches or vaults. On the other hand user-imposed loads were small, self weight was usually high enough to make wind loads relatively unimportant, and it was not difficult to ensure (without any calculation) that average compressive stresses would be well within achievable unit strengths. Except for structures subject to unusual exposure or in areas of high seismic risk, the structural requirement was therefore largely reducible to that of selecting a form whose geometric configuration was potentially stable under self-weight gravitational forces and whose proportions were adequate to avoid buckling and high stress concentrations. Yet with little more than intuition to guide the selection, the only test of a design was to build it. Thus the development of new forms was highly empirical. Sometimes this can be seen in a single building. More frequently it is apparent in a sequence of similar buildings, each going a little further in some direction than its predecessors.

The greatest single step into the unknown was that taken by Justinian's architects, Anthemius and Isidorus, in the building of the 6th century church of St Sophia in Istanbul. The entire central space was covered by a vault of interlocking part-spherical surfaces rising to a central dome some 30m. in diameter (Figure 1). The architects were professional mathematicians and probably saw this vault system largely in terms of geometry, realising that it was virtually undeformable under gravitational load provided that its supports held firm. They were less able to see how much strength and stiffness the lateral supports should be given to resist both the outward thrusts generated and possible earthquake loads. The bracing arches at ground and gallery levels marked with asterisks in Figure 2 had to be added during construction to halt outward movements that were already taking place. And on three subsequent occasions the dome and main supporting semidomes had to partly rebuilt after earthquake damage, the rise of the dome being increased on the first occasion [6,7].

In the Gothic cathedral of the 12th and 13th centuries, the ribbed vault became the spanning and space enclosing element. In successive structures designers lifted the vaults higher and higher and, at the same time, reduced their immediate supports to isolated piers to allow large areas of glazing (Figure 3). Even under the predominant gravitational load, this called for additional support to resist outward thrusts. In principle, iron ties across the springings of the vaults would have served. But lateral support was now required also against wind forces. After several mishaps, or near mishaps, that called for the addition of external props, the flying buttress was developed as an integral part of the total structural system [8].

In the early 15th century we see the outstanding example of selection of another kind. The Florentines had committed themselves half a century before to the construction of a vast octagonal dome over the crossing of their new cathedral (Figure 4). Amply strong piers surmounted by an octagonal drum had been built to carry it, and there was no reason to doubt that, once completed, it would safely stand. The problem was to build it - to ensure its stability at all stages while still incomplete. The octagonal form presented problems here that do not arise with a circular

Fig.4 (above) Florence Cathedral. © Author

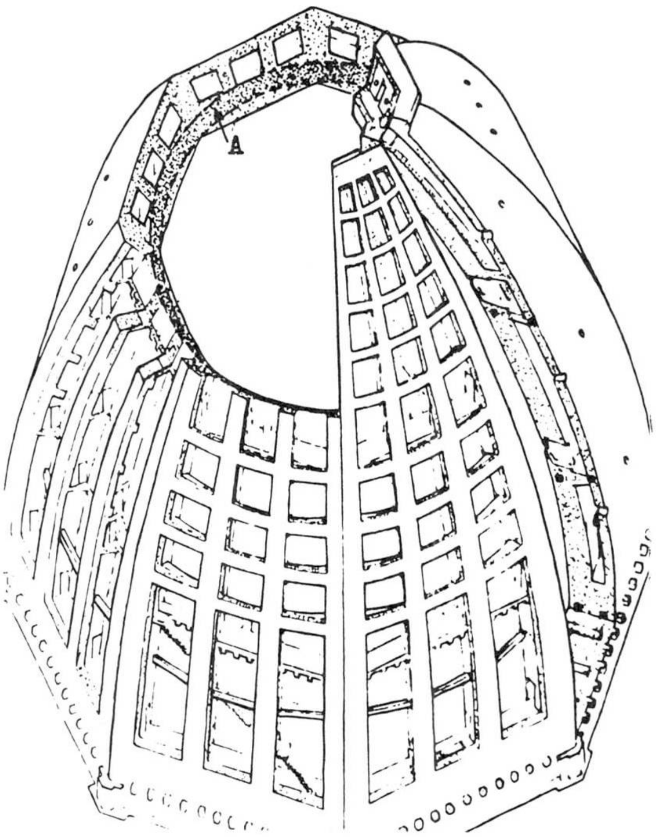


Fig.5 (right) Florence Cathedral, isometric of the dome partly cutaway to show construction. A circular dome is contained within the thickness of the inner octagonal shell as shown at A. © Author

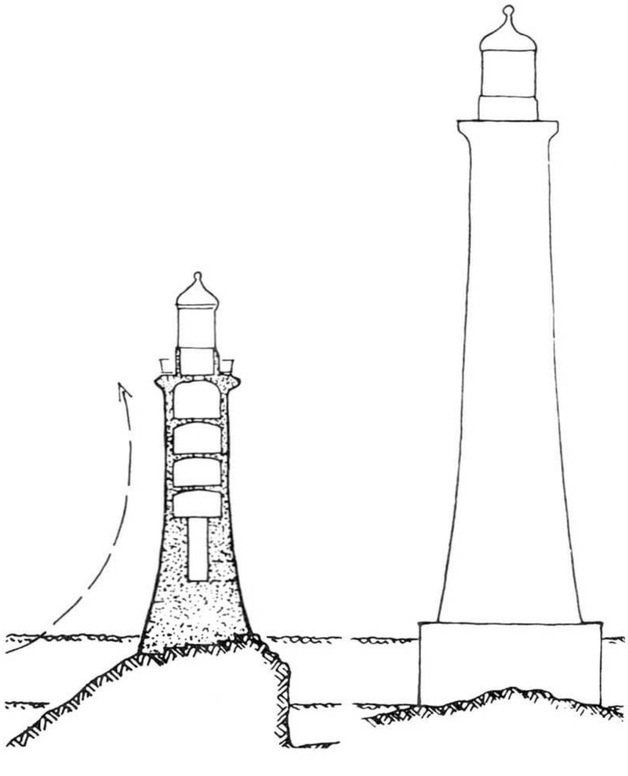

Fig.6 Eddystone Lighthouses: Smeaton left showing by dashed line the observed wave action in a storm; Douglass right. © Author

form because the centres of the sides tend to fall inwards even when there is a completed ring at the top. Previous practice - on a much smaller scale - had been to use centering for temporary support against gravity. Brunelleschi saw the difficulties of erecting and afterwards removing the huge timber frames that would be required and saw that he could entirely dispense with them if he constructed the dome as if it were, in every respect but that of surface geometry, a circular one (Figure 5) [9,10].

In all these cases consideration of the need to ensure stability played a part in the final development of the design. But interest in the space-enclosing and aesthetic qualities of the forms, and even in their symbolic connotations in some instances, was probably more important in the initial selection. Structural concerns came more to the fore in some of Leonardo's and Wren's dome designs [11], and they became dominant in Smeaton's design for the third Eddystone Lighthouse [12].

Smeaton undertook to replace the previous largely timber tower with a permanent structure of stone. It was a formidable task on account of the frequent fury of the sea. He chose to oppose the force of the waves with the enduring dead weight of masonry and selected a curved tapered profile on the analogy of that of an oak tree (Figure 6). At about the same time he carried out experiments on the forces exerted by wind and water, but he failed to appreciate the extent to which his choice of profile would lead to water being thrown up the side of the tower. The discovery that this happened was probably the first instance of the recognition of a major influence of the selection of form on loading other than self weight. It led to the selection of a modified profile for most similarly situated later towers as seen to the right of Figure 6.

## 3. THE NINETEENTH CENTURY

During the 19th century first cast and wrought iron, then steel, and finally reinforced concrete were added to the materials readily available to the designer. Their higher unit strengths, especially in tension, opened up a wider choice of structural forms. The new forms were mostly lighter than the old, so that they could span further or rise higher. This meant that loads other than self weight became more important - loads such as those imposed by wind and use. The former became particularly important on suspension bridges and the latter on railway bridges. (Neither had been of much importance on earlier arch bridges carrying only light road traffic.) And, since neither was related in any simple invariant way to the structural form, each called for explicit consideration when the form was selected. As the century progressed, this became increasingly possible through the acquisition of new data and the development of analytical tools for calculating structural responses. The chief limitations were that these tools were largely restricted to the calculation of statically determinate responses to static forces, so that wind loads and the loads exerted by moving locomotives had both to be considered as quasi static.

The influence of force on form is seen most clearly in the suspension bridge (Figure 7). The flexibility of the main chains or cables means that the designer does not have the freedom to select their profile that he has in the case of an arch. Given the span, he can choose the sag at the centre. The profile must then be that required for equilibrium with the loading, the relevant loading here being just the total self weight of

Fig.7 Clifton Bridge.
© Author

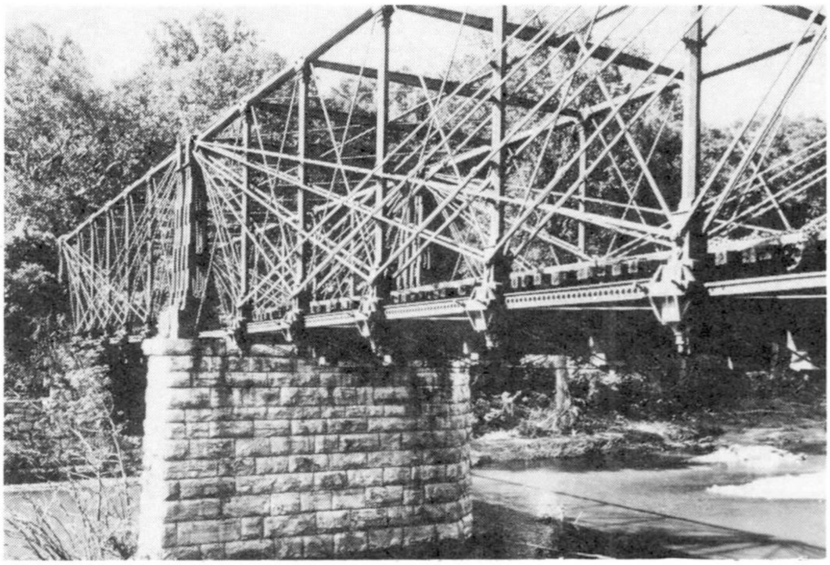

Fig.8 Savage Bridge, Md.
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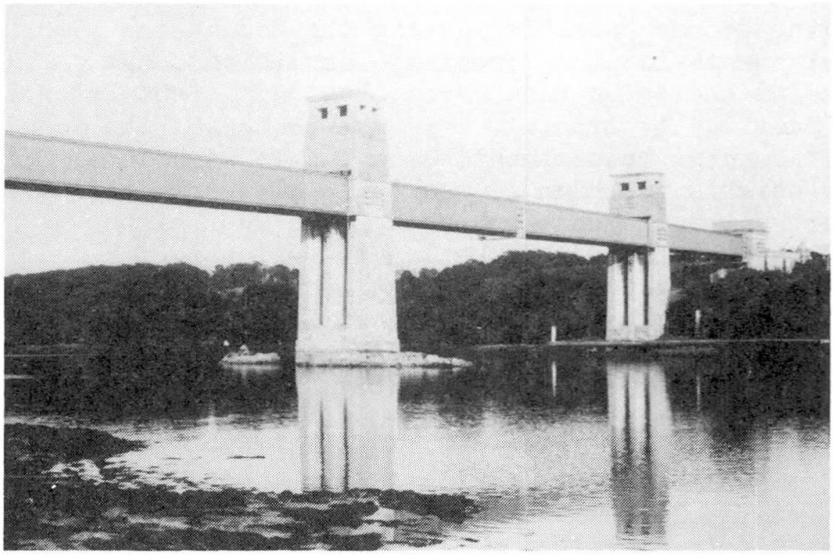

Fig.9 Britannia Bridge.
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whole span. Stiffness to resist wind and to distribute concentrated imposed loads moving over the span has to be provided by the deck. The hazard of wind induced oscillations became apparent, however, only from repeated collapses of decks that were too flexible, and the requisite stiffening remained largely a matter of judgement [13].

Increasing railway loadings led to the development of many new types of truss and girder bridge - forms which were inherently stiffer and thus preferable over most spans to the suspension bridge. While truss action remained imperfectly understood, the forms selected tended to be highly redundant. An example is seen in Figure 8. The top chord seems here to have been envisaged as the principal member, stiffened in each span by five independent sets of struts and inclined ties to assist in supporting a load moving over the span. Later, when analysis of the internal forces became possible, simple statically determinate forms were preferred. An outstanding example of the girder form was the Britannia Bridge (Figure 9). Here the final selection of the form was guided by extensive tests on models as well as calculations of the effects of the expected loads [14]. Wind was considered but not thought to call for any special provision. Longitudinal thermal expansion was, however, provided for. After the Tay Bridge collapse a static wind pressure of 2.7 $kN/m^2$ was assumed in the design of the Forth Railway Bridge and the cantilever arms were splayed in plan and transverse profile to help resist it [15].

## 4. THE TWENTIETH CENTURY

Choice has now been further widened by improvements in materials and fabrication and construction techniques and by vastly increased understandings of loads, structural responses, and the not infrequent partial dependence of load on response. In particular we are now able to consider loads like wind and earthquake dynamically and to compute structural responses involving high degrees of statical indeterminacy. Selection can now focus more of the basic choice of form, mode of action, and manner of construction. As the chosen forms are analysed, possibilities of reducing some of the loads by suitable further choices of significant parameters like stiffnesses and damping can be explored. There will be further discussion in the following papers, so a few examples must suffice here for comparison with those from earlier periods.

The tall multi-storey building became possible towards the end of the 19th century as a result of developments in steel framing, foundations, and servicing possibilities. Wind had to be considered, but sufficient lateral stiffness could readily be provided by bracing. In the choice of the form of the building as a whole, planning requirements came before structural ones. At the much greater heights to which we now build, lateral stiffness is not so easy to ensure without excessive cost. The much higher wind loads, and in some cases possible earthquake loads, call also for careful consideration of dynamic responses. Planning requirements still strongly influence the choice of overall form. But, for structural efficiency and economy, the type of framing system seen at the right of Figure 10 has had to give way to others seen further back in which the whole perimeter of the building becomes, in effect, a stiff tube [16,17]. Several recent progressive collapses have demonstrated a further need, even in the case of buildings of moderate height, to consider the effects of possible extreme loads such as local explosions and to ensure that any resulting damage will be limited in extent.

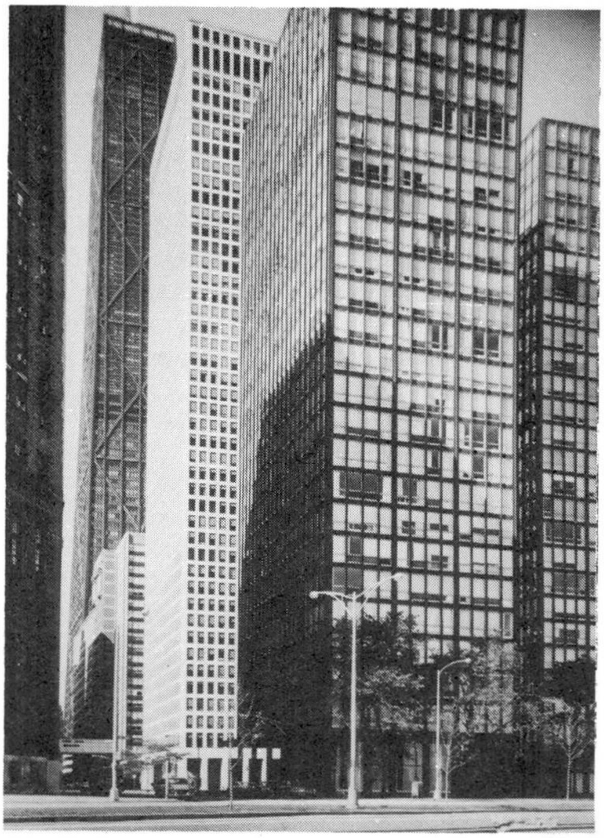


Fig.10 Lakeshore Drive Apartments, De Witt Building, Hancock Building, Chicago (right to left). © Author

Fig.11 Severn Bridge deck. © Author

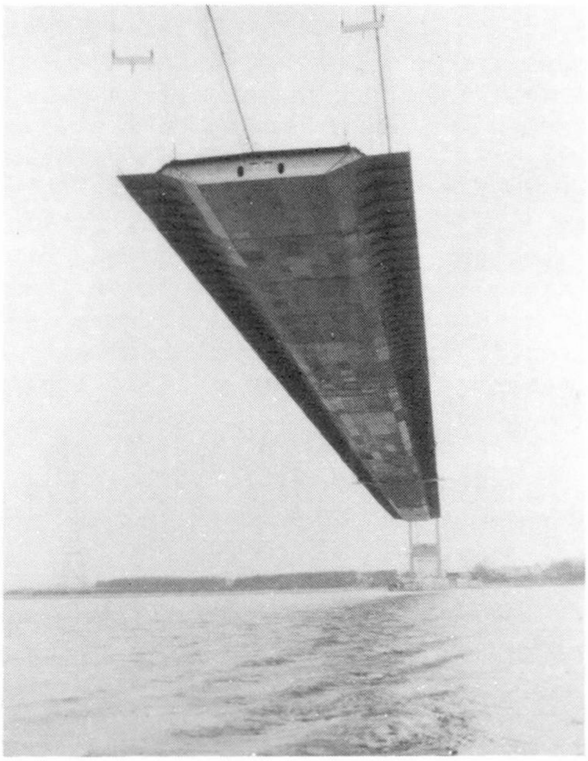
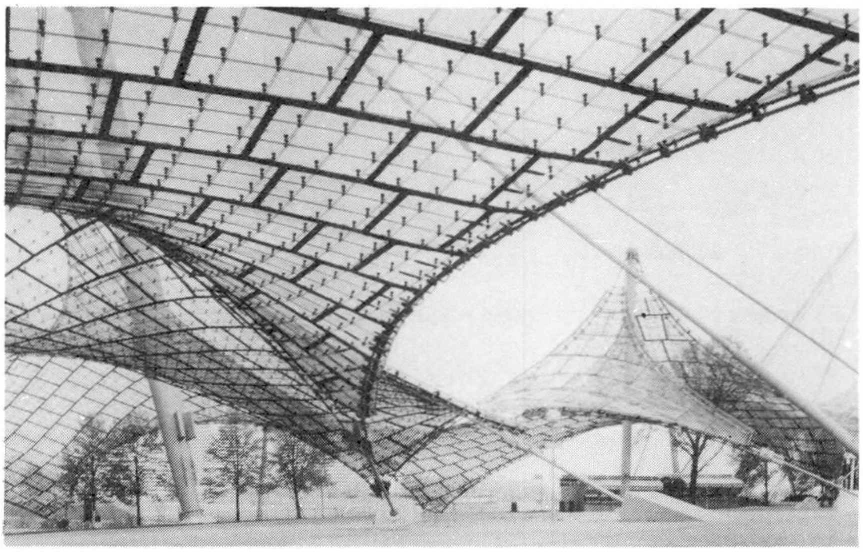


Fig.12 Tension roofs for the Munich Olympics. © Author



The dynamic characteristics of wind loading and response came to the fore with the collapse of the Tacoma Narrows Bridge deck. In the George Washington Bridge, built a decade earlier, it had been found possible to dispense with a deck stiffening truss on account of the great preponderance of the self weight of deck and cables over other forces on such a long span. Partly on this precedent, the Tacoma Narrows deck was stiffened only by shallow plate girders. Though designed to withstand static wind forces, it was twisted to destruction by a fluctuating combination of lift and drag forces. Later, in the Forth Road Bridge, torsionally stiff open trusses were used in place of plate girders and longitudinal gaps were left in the deck to help equalize pressures above and below. Then,in the Severn Bridge, a streamlined box girder was substituted for the truss-stiffened deck (Figure 11). This virtually eliminated eddies from the air flow, thereby greatly reducing the wind forces to be resisted. Supplementary damping was provided by a modified suspension system from the main cables [18].

Today, the chief counterparts of the wide-span structures considered in section 2 are shell, membrane, and cable-net forms. Membranes and cable nets are so light that wind and snow loads become the principal ones to be considered. With this lightness goes a natural flexibility, so that it becomes as important as in the case of the suspension bridge to ensure adequate stiffness. The desirability of a reasonable uniformity of stress also places constraints on the selection of geometry of surface and boundaries. In cable net roofs, stiffness is best provided by the adoption of anticlastic surface geometries which permit the prestressing of the cables in one direction against those in the other if adequate anchorage is provided along the boundaries (Figure 12). In some small membrane structures (tents) it is similarly ensured. Alternatively the whole membrane may be prestressed by internal inflation. Here, particularly in the case of the single-skin pneumatic structure, a man-made load - the internal pressure - becomes an element of the structure, though this might almost be said of all prestressing forces. Moreover it will significantly affect the external dynamic wind forces - an interesting situation which we are still exploring.

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I

## The selection of structural form to resist wind

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**At the time this report was printed Professor Davenport's paper had not been received in the UK.**

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# The influence of thermal forces on the structural form

L'influence des forces thermiques sur le système et la forme d'une structure

Der Einfluss thermischer Einwirkungen auf das System und die Form eines Tragwerks

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**SUMMARY**
In this report it.is summarized how the design of structures is being influenced by thermal effects. At first the changes of the material properties under increased as well as low temperatures are being described. Further on remarks are given how imposed deformations caused by thermal effects may be decreased respectively avoided or be taken into consideration by constructional and design measures.

**RESUME**
Cette contribution montre l'influence des effets thermiques sur le projet de bâtiments. Le changement des propriétés des matériaux de construction est présenté pour des températures et basses. Des indications sont données sur la façon d'annuler les dérformations imposées, produites par les actions thermiques: ceci peut être obtenu par la prise en compte dans le calcul et par des mesures constructives.

**ZUSAMMENFASSUNG**
Der Beitrag gibt einen Üeberblick, inwieweit der Entwurf von Bauwerken durch thermische Einflüsse beeinflußt wird. Es werden zunächst die Veränderungen der Baustoffeigenschaften unter erhöhten sowie tiefen Temperaturen beschrieben. Dann wird gezeigt, wie Zwängungskräfte aus thermischen Einwirkungen vermindert bzw. gänzlich vermieden oder durch entsprechende konstruktive Maßnahmen aufgenommen werden können.

## 1. GENERAL

In structures and structural elements thermal forces are resulting mainly from two reasons
- climatic conditions of the location of the structure and
- special thermal influences connected with its use.

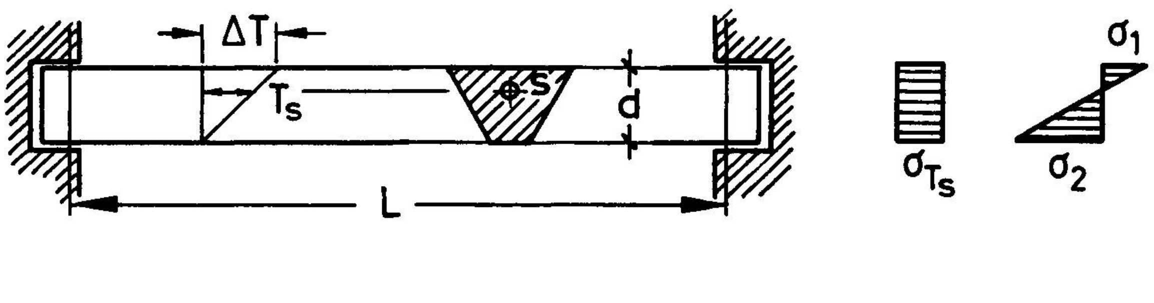
Extraordinary thermal conditions, as e.g. fire, need special considerations and measures.

Temperature changes in a structural element in relation to its initial temperature during construction either lead to deformations or to internal forces - if the thermal deformations are restrained. These internal forces are in equilibrium to the imposed thermal deformations, their magnitude thus depends on the resistance of the structure opposed to the deformations. This resistance is governed by the structural stiffness and the material properties of the whole involved structure. For concrete elements the stiffness depends among other things directly on the amount of external loads, the formation of cracks and time dependent effects, such as creep. By changing the stiffness of the members in a concrete structure the load depending internal forces may at least be redistributed, restraining forces in contrary are changed with respect to their amount.

For the estimation of thermal effects in structural elements it is suitable to pass over to the stresses hereby caused:

We differentiate between restraining and residual thermal stresses:

Restraining thermal stresses are caused by imposed deformations in the structure and occur only in statically undetermined systems (hyperstatic structures). They may be deduced directly from linear temperature changes. The sum of restraining stresses in each cross-section is generally different from zero; just as load stresses, restraining stresses may be summarized to internal forces (thermal load effects) and may be treated in design work equal to ordinary load effects (Figure 1a).




Equal temperature distribution over the whole length L, linear temperature gradient $\Delta T$:

axis elongation $\Delta L \cong \alpha_T \cdot T_s \cdot L$, curvature $1/K = M/E \cdot J = (\alpha_T \Delta T)/d = \text{const}$

longitudinal restraint: $\sigma_{Ts} = \alpha_T \cdot T_s \cdot E$

flexural end-restraint: $\sigma_{1,2} = \pm\, \alpha_T \cdot \Delta T \cdot E \cdot J/d \cdot W_{1,2}$

rectangular cross-section: $\sigma_{1,2} = \pm\, 0.5 \cdot \alpha_T \cdot \Delta T \cdot E$

Fig. 1a Linear Temperature Strain Distribution and Restraining Thermal Stresses (Ideal Elastic Material)

Residual thermal stresses occur under non-linear temperature in the cross-section and are characterized by the fact that their sum in each section is zero. They may, however, cause deformations of the structure. In hyperstatic structures these deformations may lead to residual stresses and thermal load effects (Figure 1b).

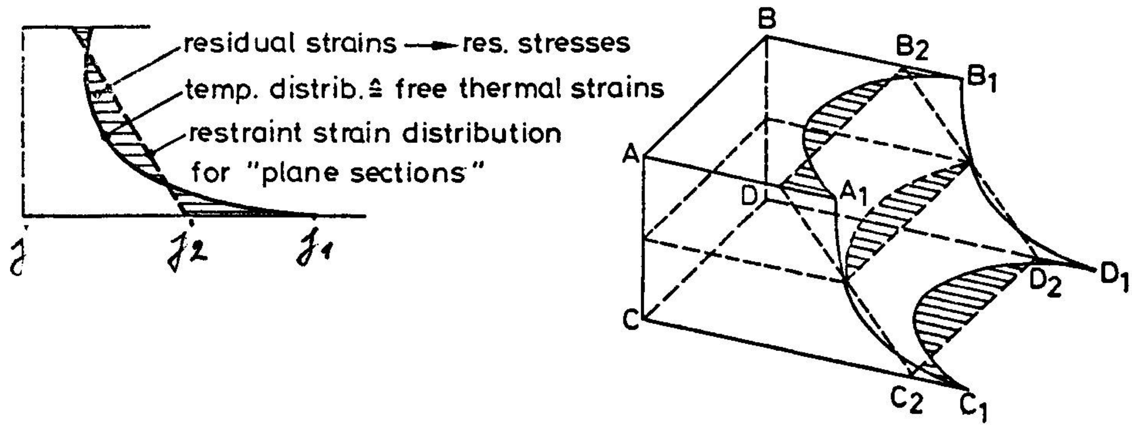



Fig. 1b Nonlinear Temperature Strain Distribution; Residual Thermal Stresses

The following table gives limiting values of temperatures; in general design is done according to these limiting values. For concrete structures the temperature in construction (column 3) corresponds approximately to the temperature of the fresh concrete, the rise of temperature by hydration is neglected. The temperature differences (column 4) refer to the surface temperatures of the unprotected structure [1, 2].

| influence | region | temperature in construction | temperature differences under service conditions |
|---|---|---|---|
| 1 | 2 | 3 | 4 |
| environmental conditions | Central Europe | + 15°C | ± 30°C |
| | Polar Region | ∼ 0°C | + 15°C <br> ≦ - 50°C |
| | Subtropical Regions/Tropics | + 35°C | + 35°C <br> - 35°C |
| thermal effects by the use | up to | | ± 200°C |
| fire | | | ≧ + 700°C |

Table 1

## 2. INFLUENCE OF TEMPERATURE ON THE MATERIAL PROPERTIES

### 2.1 Steel

The change of the strength characteristics of usual steels under elevated temperature may be taken from Figure 2, 3 and 4. Figure 5 shows E-modulus and ultimate strain in failure vs. temperature of reinforcing steel.

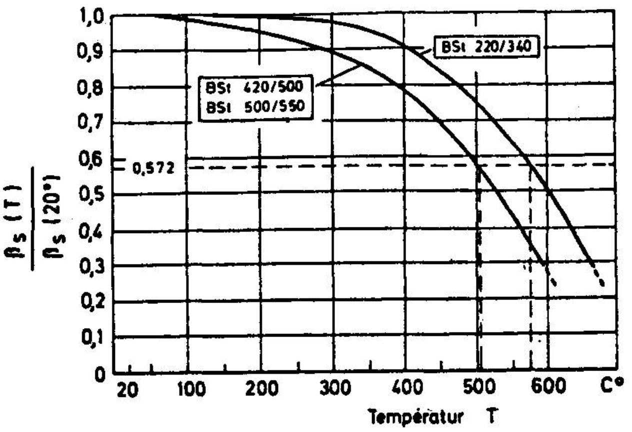



Fig. 2 Yield Stress of Ordinary Reinforcing Steel vs. Temperature

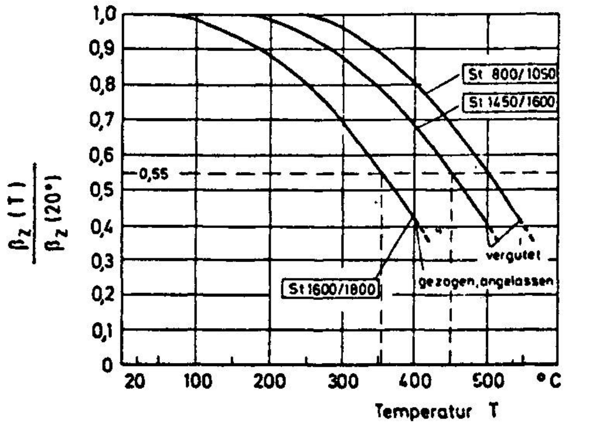



Fig. 3 Tension Strength of Prestressing Steel vs. Temperature

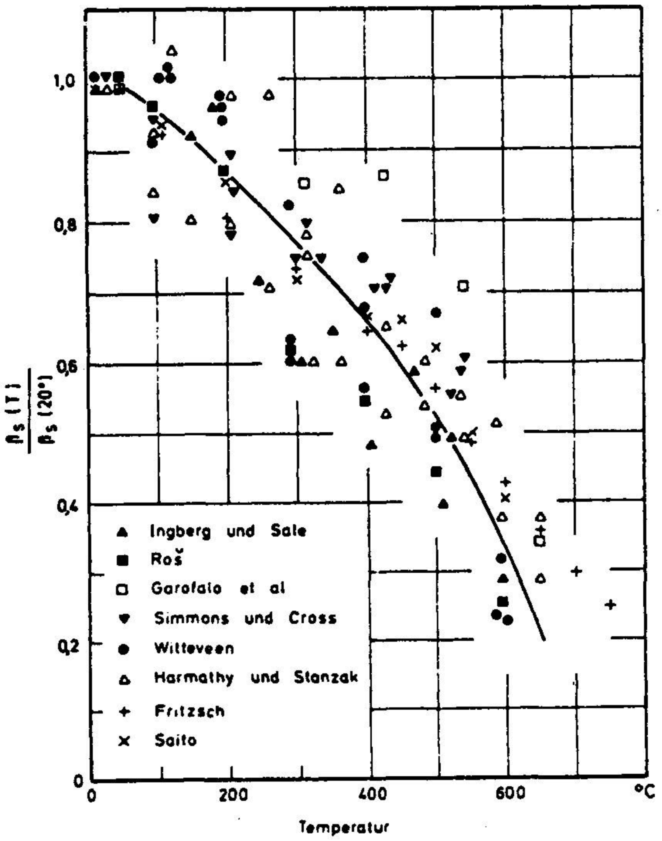



Fig. 4 Average Yield Stress of Structural Steel vs. Temperature

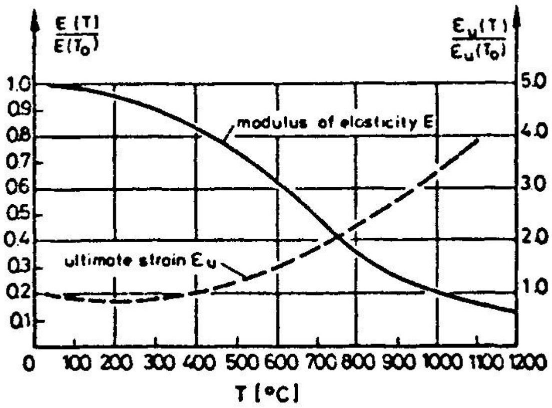



Fig. 5 Modulus of Elasticity and Ultimate Strain of Reinforcing Steel vs. Temperature

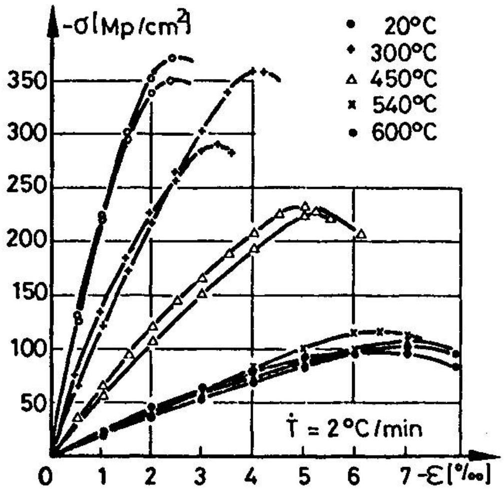



Fig. 6 Stress vs. Strain of Normal Concrete under Elevated Temperatures

If cold-deformed, respectively self-hardening steel is extremely heated, it shows different properties after cooling:

In case of slow cooling after heating of $\geq$ 400°C, self-hardening steel regains approximately its original material properties; a sudden cooling leads to embrittlement. Under certain conditions, cold-deformed steel does not or only partly regain its properties as a change in the texture may occur.

Under the influence of low temperatures (-180°C) ordinary steel becomes brittle, especially in the area of nicks. The material properties including the E-modulus increase slightly compared with the values under normal temperature, only the ultimate strain in failure decreases clearly [30, 31].

The thermal expansion of steel can be determined in a satisfactory way in the area of -200 up to +500°C with $\alpha_T = 10^{-5}$.

## 2.2 Concrete

The compressive strength of concrete decreases extremely under elevated temperatures; however, the loss of strength under unique heating is very low for temperatures $\leq$ +200°C, as it may happen under service conditions (Figure 6). Repeated heating on temperatures > 100°C leads even after 20 cycles to a significant loss of strength (> 20%), whereby simultaneous wetting and drying are acting critically [4]. The time dependent deformations (creep, relaxation) occur accelerated and enlarged compared with standard temperature [3, 4, 5]. Figure 7 shows exemplary the relaxation behaviour of concrete under heating with restrained elongation, Figure 8 shows the scatter band of the thermal expansion [5].

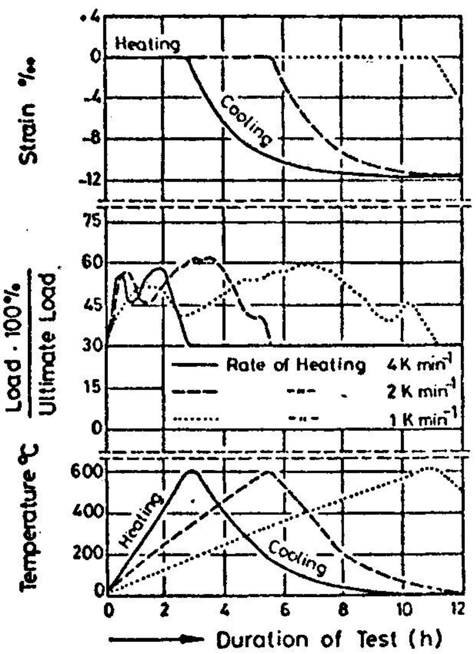



Fig. 7 Restraining Forces of Concrete Specimens with Longitudinal Restraint under Increasing Temperatures; Initial Loading 30% of Short-Time Strength

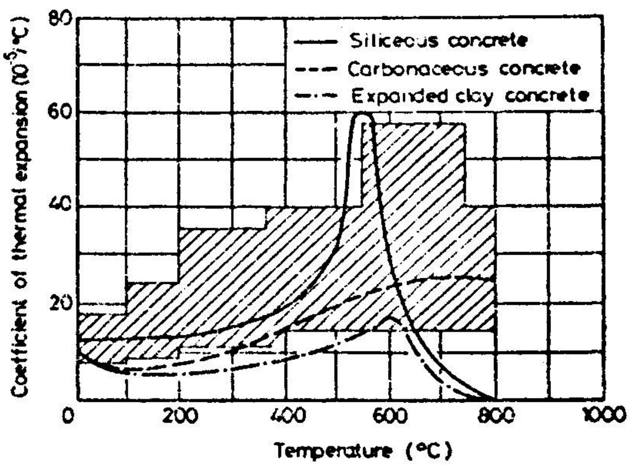



Fig. 8 Coefficient of Thermal Expansion of Concrete for Various Aggregates [3]

Concrete that is exposed to high temperatures for some time, especially > 100°C, drains progressively totally and therefore loses its active protection against steel corrosion [6, 7].

Figure 9 shows the stress-strain relation of concrete (at ambient temperature and at -170°C) that was stored in water respectively under standard climate 20/65 conditions until testing after approximately 90 days. It shows that the compressive strength and the E-modulus increase depending on the state of humidity under low temperatures. Concrete saturated with water reaches with low temperature a comparatively still higher compressive strength; however, a high moisture content of the concrete causes with cyclic freezing and thawing a significant loss of strength (Figure 10).

Figure 11 shows comparatively the thermal strain of prestressing steel, normally stored concrete and water saturated concrete. Under cooling cycles prestressing steel as well as reinforcing steel and normally stored concrete show quite linear shortening, but steel shows obviously the higher deformations under the same temperature decrease. In reinforced structural members therefore "self-stressing" occurs. Water saturated concrete reacts in cooling down cycles significant according to the properties of ice [8, 9].

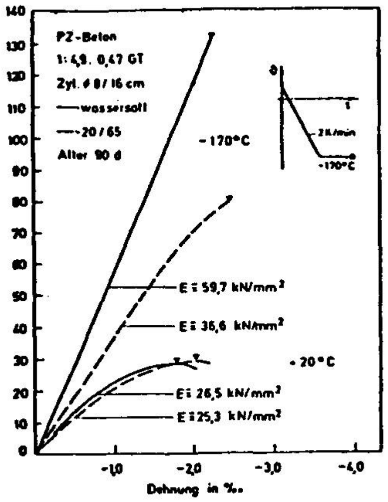



Fig. 9 Stress vs. Strain of Concrete under +20°C and -170°C; Water Saturated and Normally Restored

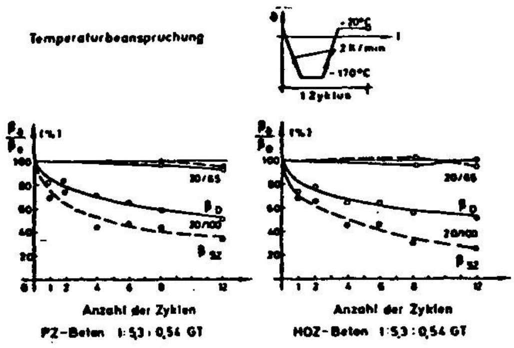



Fig. 10 Influence of Humidity on Residual Strength of Concrete after Cyclic Low-Temperature Treatments

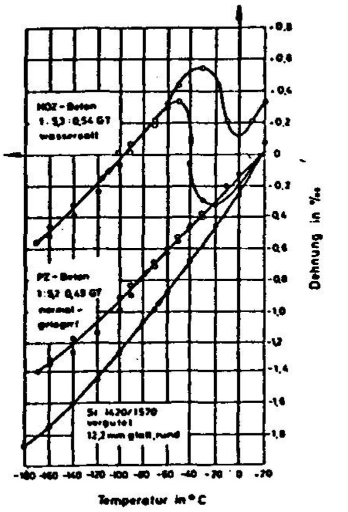



Fig. 11 Strain vs. Temperature of Water Saturated and Normally Stored Concrete and Prestressing Steel

## 3. RECOMMENDATIONS FOR CONSTRUCTION UNDER EXTREME AMBIENT TEMPERATURES

### 3.1 Steel Structures

Erecting of steel structures is in a wide range independent of the ambient temperature conditions. This is especially valid for the "hot countries". Temperatures lower than approximately +5°C may in contrary lead to embrittlement of steel and require some additional caution measures: Prestressing should not be done with temperatures below 0°C, unloading and handling of steel members must be done carefully in order to avoid cracks or brittle failure, welding may also lead to difficulties and bad results.

### 3.2 Concreting with Cold Weather

With cold weather the fresh concrete has to be placed at a minimum temperature of ≧ 5°C due to the delay of hardening and the possibility of lasting influence on the concrete properties. If the air temperature decreases below -3°C a temperature of > 10°C has to be aspired by physical measures for the fresh concrete [10].

### 3.3 Concreting with High External Temperatures

Temperatures of more than 30°C of the fresh concrete can lead to loss of strength of the hardened concrete; usually a temperature of ≦ 30°C is required for the fresh mixed concrete. The observance of this temperature limit is in subtropical and tropical countries not possible without additional measures [2, 10]. Some of these measures are:

- white paintings for cement bunkers and vehicles for ready mixed concrete
- spraying the gravel and sand boxes with water; the cold due to evaporation reduces the temperature of the fresh concrete for 2 - 3°C

- cooling of the mixing water, addition of ice.

Another danger for the hardened concrete is extreme sunlight, hot wind and low humidity of the air; the concrete has to be protected against fast drying; sufficient supply with humidity has to be guaranteed (curing).

### 3.4 Plastics

For the hardning of plastics, e.g. artificial resin adhesives, the temperature of the structure and the external temperature are of importance as is known; although the temperature of the structure is often higher than the atmospheric temperature in the northern regions occur temperatures of the structure of less than 5$^{o}$C, so that a satisfying hardening process of the artificial resin can not be expected. In this case it is also not sufficient to mix the adhesive under increased temperature - e.g. in a heated room - as the adhesive adopts after application immediately the temperature of the structural element. In the last time special methods were successfully applied: The joints of segmental bridge girders were warmed up with heating wires placed in the glue. By this it was possible to extend the period of construction in spite of the cold weather and to limit the hardening process of the resin on a practicable time [11].

Resin mortars and similar materials as used e.g. for glueing segmental prestressed girders show accelerated hardening process under high ambient temperatures. This phenomenon has to be taken into account because of its influence on the time of workability of the mortar in hot countries.

## 4. DESIGN FOR THERMAL LOAD EFFECTS FROM ENVIRONMENTAL CONDITIONS

### 4.1 Steel Structures

In general temperature dependent stresses in steel structures have to be considered for the analysis under service conditions. Whether and to what extent imposed deformations and thermal load effects may be neglected in a (plastic) ultimate limit state design depends from the kind of the structure and its load bearing behaviour. The neglection of thermal load effects in structural elements being mainly stressed by compression is not in principle permitted [12, 13].

For composite steel structures the different thermal conductivity of steel and concrete has to be considered; for composite steel bridges e.g. it is usual to treat the following thermal load cases:

- constant change of temperature in the whole bridge cross-section
- linear temperature gradient over the whole cross-section (surface of the bridge deck warmer than bottom flange)
- temperature difference between concrete bridge deck and the steel structure.

### 4.2 Concrete Structures

Check Under Service Conditions:

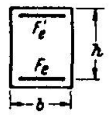
If the thermal conditions influence the state of stress in a hyperstatic structure in an unfavourable sense, then the maximum values of the thermal load effects have to be considered. These effects have to be calculated using stiffness values on the safe side. With structural elements under bending with and without tensile forces the decrease of stiffness due to cracking may be considered (see the following table) [14, 15].

If on the other hand the state of stress is influenced by the thermal conditions in a favourable way, thermal effects may generally be neglected, as for the behaviour of the corresponding structural elements the load condition "external load without simultaneous temperature influence" is relevant.

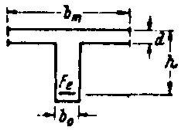
With reinforced concrete or prestressed concrete structural elements being subjected to certain thermal effects for a long time it is generally useful to consider the favourable influence of creep. However, it must be considered that the decisive load conditions often already coincide with the origin of thermal effects and therefore the decay due to creep does not become relevant.

The following table gives upper boundary values for the real flexural stiffness of reinforced concrete structural elements in dependence of normal force and degree of reinforcement:

| steel quality | axial force n | percentage of reinforcement $\mu$[%] | $\chi = \frac{K_B}{E_b J}$ |
|---|---|---|---|
| BSt220/340 | all n | all $\mu$ | 1.0 |
| BSt420/500<br>BSt500/550 | n<-0.15 | all $\mu$ | 1.0 |
| | $\lvert n \rvert \leq 0.15$ | $\mu \leq 0.6$ | 1.0 |
| | | $\mu > 0.6$ | 0.65 |
| | n>+0.15 | all $\mu$ | $0.2+6(\mu+\mu')$ |




$n = \frac{N}{bh\beta_R}$; $\mu = \frac{F_e}{bh}$; $\mu' = \frac{F_e'}{bh}$




$n = \frac{N}{b_l h\beta_R}$; $\mu = \frac{F_e}{b_o h}$; $\mu' = \frac{F_e'}{b_o h}$

$b_l = b_m \frac{d}{h} + b_0 \left(1 - \frac{d}{h}\right)$

Table 2 Effective Stiffness in Bending $K_B$ of Reinforced Concrete Structural Members depending on the Actual Axial Force and the Percentage of Reinforcement $\mu$

Design procedure:
In general an ultimate state design is required; therefore the relevant load effects are determined according to equation (1):

$$N_U = \nu_Q \cdot N_Q + \nu_{th} \cdot N_{th} \qquad (1)$$

For reinforced concrete structural elements in bending, the safety factor $\nu_{th}$ for thermal load effects may be assumed as 1.0, as with increasing loads and deformability of the structure the thermal load effects - depending on imposed deformations - decrease. With structural elements being mainly stressed in compression a significant decrease of restraining forces near the ultimate state of stress does not have to be expected; therefore in these cases equation (1) should be verified with $\nu_{th} \cong 1.5$.

However, in addition the total state of stress under service conditions should be examined using the values of table 2 in order to avoid unjustifiable crack formations, according to equation (2):

$$N = N_Q + N_{th} \qquad (2)$$

## 5. INFLUENCE OF THERMAL LOADS ON THE STRUCTURAL FORM

### 5.1 General Remarks

Structural elements being subjected to extreme and significantly changing temperatures must be free to deform or able to accommodate the imposed deformations. The formation of cracks due to thermal effects under service conditions may be accepted for reinforced concrete structural elements only in exceptional cases when cracks are neither for the load bearing capacity nor for the function of the structure of importance. Therefore in most cases it is necessary to retain the uncracked state. In hot countries the fluctuations of the external temperature due to the day/night cycle are very serious; in general the temperature gradients caused hereby in walls and roof structures have to be considered taking into account big temperature differences: The surface temperature may reach 70°C and more under intensive exposure to sunlight in subtropical areas, whereas the temperatures during the nights may reach more than 30°C lower. The deformations and thermal load effects in the structural elements reach three- up to fourfold values compared with the conditions in Central Europe; so the thermal effects exceed by far all ordinary load effects [2, 29].

Additional difficulties occur if the structure is not only subjected to such extreme climatic conditions but also has to take up hot liquids. With concrete structures the problem to guarantee no cracks becomes special importance. Especially in such cases thermal load effects have to be avoided if possible; a sufficient number of contraction joints in connection with statically determined structures can mostly be successful. The price of such a structure was often found smaller than with a jointless structure and perfect consideration of the thermal loads [2].

## 5.2 Joints

Structural elements between expansion joints shall be able to move uniformly to all sides, if possible; stiffening components as staircases or elevator shafts therefore should be arranged in the middle between two joints respectively between joint and end of the structure. Joint spacing a and gap of the joint b have to harmonize with the expected motions of the structure and the restraining forces as e.g. friction on the soil or stiffness respectively deformability of the relevant structural elements. As far as risk of fire has to be considered, the gap of the joint b should be chosen as

$$b = a/800 \quad \text{up to} \quad b = a/600$$

Joint spacings in buildings of more than 40.0 m require in general special investigation [10, 16].

Designing of expansion joints in structures under fire protection aspects has to consider that even after a long period of usage of the structure unrestricted possibility of motion must exist in case of fire, but the passage of fire or hot gases through the joint must absolutely be avoided. In this connection it must be taken into account that maintenance work on expansion joints is mostly omitted, in some cases even impossible. The penetration of dust or other particles must be avoided and aging of the filling and covering materials of the joints should not limit their function even under permanent stressing by movements of the building [15, 16, 17].

Figure 12 shows a joint designed under fire protection aspects.

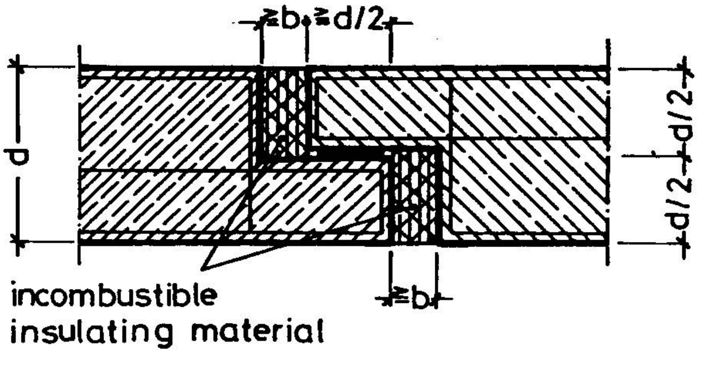



Fig. 12 Proposal for the Design of an Expansion Joint with High Fire Resistance Properties

### 5.3 Design Considerations

If it is not possible to avoid imposed deformations by thermal effects, e.g. if joints and in consequence differential settlements can not be accepted, a very careful coordination of the over-all design has to be executed with the aim to reach an optimization of the dimensions of the structural elements (e.g. wall and roof thickness) with respect to thermal load effects, shrinkage etc. and ordinary load effects.

It is often favourable to avoid influences of great temperature differences by means of efficient insulation measures; as far as the local conditions allow it, the possibility of an external covering with earth should be taken into account; this is probably the cheapest solution.

The use of prefabricated structural elements offers several advantages to avoid thermal load effects. As far as reinforced concrete structural elements are concerned the curing can be executed precisely at the manufacturing plant, whereby also undesired shrinkage effects can be excluded. Suitable supports can guarantee the flexibility of each structural element under temperature changes and thus prevent thermal load effects.

With steel structures in buildings or other engineering structures the protective effect of insulations is looked for; however, the remaining temperature elongations can mostly be absorbed by the whole load bearing structure. Movable supports, slotted hole connections etc. are hardly no more produced [13].

An exceptional case should here be pointed out, which is characteristic for the additional difficulties in hot countries: Overflow pipes made of reinforced concrete are normally not coated inside in moderate climate. In the Orient, however, where the sewage temperature is relatively high and wash water has to be economized due to water shortage, such a coating is necessary - mostly on the basis of tar-epoxy resins - as the microbes producing aggressive acid multiply best at water temperatures of +30°C [2].

The architectural design can also be useful with respect to a reduction of difficulties raised by temperatures:

Ventilated roof structures produce a clear improvement of the ambient temperature inside the building without expensive air conditioning installations, especially if the roof structure is projecting over the facade, giving shadow. Furtheron a shadowy covering of the external facade - e.g. with a prefabricated concrete network being nevertheless light-transmissive - will in many cases be just as much of advantage. It must be pointed out that these basic ideas can even be found from ancient times in the original typical buildings in subtropical and tropical zones.

## 6. EXAMPLES FOR STRUCTURAL DESIGN

The selection of the structural form of any building with respect to thermal forces is connected in a very sensitive way with the basic assumptions for its design and verification. More difficulties in this respect arise with concrete structures; therefore two examples taken from this field of application shall finalize this report, giving perhaps some practical recommendations!

### 6.1 Design of a Silo for Thermal Load Effects

Silos for cement clinker have to be designed not only for the usual loads (dead load, wind, pressure of the filling material etc.) but also for thermal load effects. The medium filling temperature of the clinker is ranging between 100°C and 200°C depending on temperature fluctuations during process of the production of clinkers. The actual clinker temperature depends besides others primarily on type and efficiency of the cooling system being subsequently added to the furnace and length of transportation distance to the feed opening. The hot clinker transfers part of its heat on the air inside the silo and part on the structural elements of the silo by means of conduction, convection and thermal radiation.

In the area below the level of the filling material the silo wall is being heated through the direct contact to the clinker (heat conduction). Above the filling material the heat flow consists on the one hand of a convective part, whereby the clinker transfers the heat by the air on wall and roof of the silo; on the other hand the heat flow results from the radiation of the clinker.

It is obvious that the maximum temperature gradient in the silo wall adjusts above the level of the filling material, resulting from radiation and convection. The air temperature above the filling material increases with rising level of the filling material and consequently the temperature gradient in the silo wall increases.

Below the level of the filling material the temperature gradient in the silo wall produced by direct contact with the filling material decreases quickly through a nonsteady heat conduction, whereby the cooled filling material next to the silo wall acts as an insolation to the more heated cement clinker inside.

In principle, the temperature conditions for the area above and below the level of filling material can be approached by a heat balance calculation [18, 19, 20, 21]. But this is often too difficult. For the designing procedure some recommendations for approximate temperature distributions are needed. Figure 13 and 14 show datas gained theoretically but examined by measurements in a clinker silo, which can be taken as basis for further calculations [21, 22]. In the area below the level of the filling material it may be assumed a linear decrease of the temperature gradient and of the temperature in the middle plane of the wall up to the bottom of the silo.

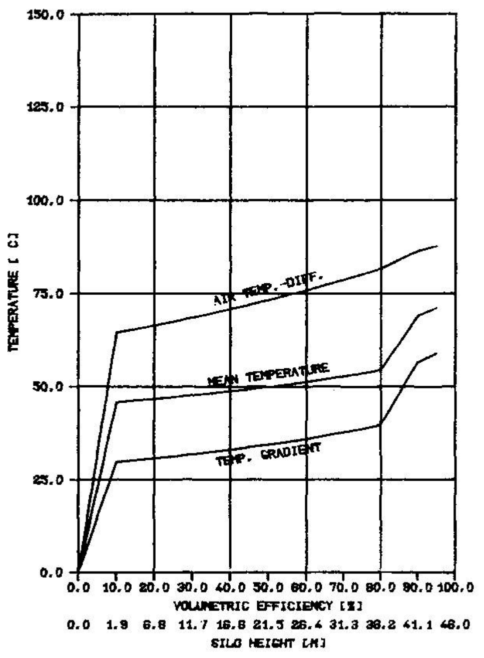
## 6.2 Design of LNG-Tanks with Respect to Cracking

The design of LNG-tanks is significantly influenced through the special importance of the thermal load case. Under service conditions the thermal load is being kept low by means of suitable insolations and operation devices. In case of a catastrophe one has to rely on the fact that by sufficient reinforcement progress of cracks and crack-width remains restricted with respect to three requirements:

- "Through-and-through cracks" may never penetrate the cross-section of the wall in order to avoid leakage of the tank.
- Although there are no signs that LNG endangers the prestressing steel by corrosion and although even under sudden cooling and impact loading the properties of the steel do not become unfavourable, immediate contact of the LNG to the prestressing steel should be avoided.
- A thermal loading by a cold wave, including a liquid impulse, must not lead

Quantitiy of Filling: 80.0 [$m^3$/h]
Dedusting: 10000.0 [$m^3$/h]

Filling Temperature: 175.0 [C]
Free-Air Temp.: 20.0 [C]




Quantity of Filling: 80.0 [$m^3$/h]
Dedusting: 10000.0 [$m^3$/h]

Filling Temperature: 175.0 [C]
Free-Air Temp.: 10.0 [C]

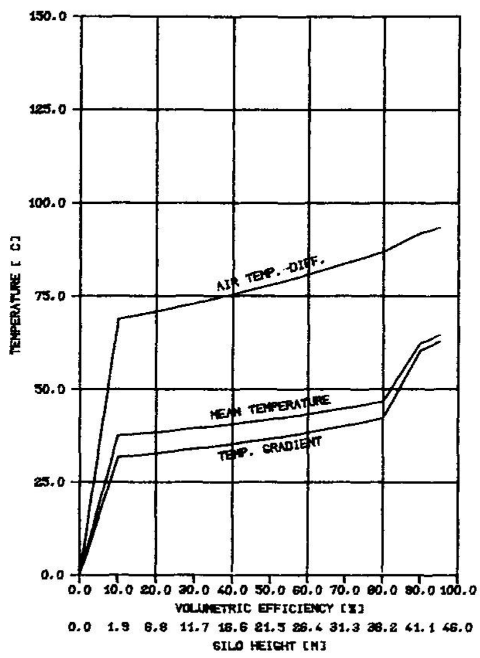



Fig. 13, 14 Temperature Distributions vs. Level of Hot Filling Good

to a brittle failure and all cracks should close again; therefore the prestressed and passive reinforcement as well must not exceed also in these cases the elastic portion.

Some aspects concerning the crack behaviour and the influence of very low temperatures are being discussed in the following:

The different coefficients of thermal contraction of steel and concrete were already pointed out. With usual material pairing theoretical thermal residual stresses in the range of 80 up to 150 N/mm$^2$, possibly higher, occur in the steel under -170°C compared with the temperature of 20°C [23]. This is about 10% of the yield stress of prestressing wires, which increases itself for 15 ... 25%. This self-prestressing effect, however, charges the bond. Furtheron it should be examined, whether instead of the load case "prestressing" before creep and shrinkage the minimum filling level in the LNG-tank under low temperature becomes the critical load condition for the concrete compression zone in the wall (including the thermal pressure stresses acting synonymously with the prestressing forces) [24].

The required prestressing forces rise progressively with the required minimum thickness x of the concrete compression zone; a remaining compression zone under bending and tension even under extraordinary conditions is essential in order to avoid leakage of the tank. Sometimes it is requested that the cracks may not reach the prestressing steel, that means $x \leqq 0.5$ h (e.g. with centric tendons). The prestressing force $F_T$, being necessary in addition to the hydrostatic conditioned prestressing force $F_N$ to guarantee a sufficient hight of the compression zone is estimated between $F_T$ = 0.6 MN/m$^2$ for x = 0.3 h and $F_T$ = 2.8 MN/m$^2$ for x = 0.5 h [25, 26]. An increase of prestressing seems to be more economic than an enlargement of the passive reinforcement in this respect.

Ordinary reinforcing steel may not be used as passive reinforcement for LNG-tanks; steel with high yield strength up to 1500 N/mm$^2$, e.g. heat-treated, ribbed wires with diameters 10 up to 16 mm, is recommended. Its design stress should not exceed 500 N/mm$^2$ under normal temperature respectively $\sim$ 70% of the yield stress under low temperature [27].

In most cases steel of this kind shows clearly smaller "related rib areas", which may produce difficulties for the bond behaviour. In addition bond increases under low temperature less than the compressive strength of concrete and is synonymously preoccupied by the self-stressing effects, therefore crack distance and crack-width are expected to be greater under low temperature than estimated under standard temperature values [23].

In connection with the problem of bursting of an inner LNG-tank by a brittle failure the question of crack propagation in prestressed tank walls occurs. For prestressed concrete such a "zipping open"-effect can be excluded if the remaining prestress after fracture of one tendon is still able to avoid an advance of the crack; that means the neighbouring tendons must be able to take up the load of the broken one by bond and to carry it within reasonable limits and reconcile the crack [28]. Sufficient passive reinforcement in good bond and tendons in not too far distance are therefore necessary conditions for suitable behaviour of the structure even under extraordinary conditions.

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I

# Reliability of Snow Roof Load Assessment

Fiabilité des hypothèses de charges de neige

Zuverlässigkeit bei der Schätzung von Schneelasten

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**SUMMARY**
The paper deals with uncertainties regarding roof snow assessment according to present codified procedures. It comments upon the statistical calculation of annual maximum snow depths with different mean return periods for a given site, upon the determination of the specific gravity of snow on statistical snow accumulation factors and the safety of the structures against snow load on a second moment format.

**RESUME**
Les charges de neige admises dans les codes actuels ne correspondent pas toujours à la réalité. L'article traite des valeurs statistiques de profondeur de neige maximum qui peuvent se rencontrer en un endroit donné, pendant une certaine période. D'autres paramètres sont pris en considération: poids spécifique de la neige, facteur d'accumulation de la neige, sécurité des structures vis-à-vis des charges de neige.

**ZUSAMMENFASSUNG**
Der Beitrag behandelt die Unsicherheiten einer Schätzung von Dach-Schneelasten aufgrund der heutigen Richtlinien. Die Autoren kommentieren die statistische Berechnung der maximalen jährlichen Schneehöhe mit verschiedenen mittleren Wiederkehrperioden für ein bestimmtes Gebiet, die Bestimmung des spezifischen Gewichtes von Schnee aufgrund statistischer Angaben, die Effekte von Schneeverwehungen ab Dächern, die Ansammlung von Schnee und die Sicherheit von Tragwerken bezüglich Schneelasten.

## 1. STATISTICAL DISTRIBUTION OF SNOW DEPTH ON THE GROUND

The statistical analysis makes use of the annual maxima of snow depth on the ground. They are considered to be independent and represent a random variabile, not a time stochastic process.

The annual extremes of snow depth for structural design are defined by their mean return period $\bar{T}$, in years, within a usual range of about two years (i.e.mean of annual maxima) and one hundred years. A probability of exceeding in one year corresponds to thease values:

$$P_{1\ year}(>) = \frac{1}{\bar{T}}$$

and in N years, in the hypothesis of annual maxima independence:

$$P_{N\ years}(>) = 1-(1-\frac{1}{\bar{T}})^{N} \qquad (1)$$

that is:

| $\bar{T}$,years | 2 | 1o | 2o | 3o | 5o | 1oo |
|---|---|---|---|---|---|---|
| $P_{1year}(>)$ | o,5 | o,1 | o.o5 | o.o33 | o.o2o | o.olo |
| $P_{in\ 3o\ years}(>)$ | 1 | o.958 | o.785 | o.638 | o.455 | o.26o |
| $P_{in\ 5o\ years}(>)$ | 1 | o.995 | o.923 | o.816 | o.636 | o.395 |
| $P_{in\ 1oo\ years}(>)$ | 1 | o.999 | o.994 | o.966 | o.865 | o.634 |

Extreme values distributions for maxima of type I, or Gumbel, and of type II, or Fréchet, as well as lognormal distribution, Pearson type III, etc., are used in statistical analysis of annual of snow depth. The higher the values for $\bar{T}$, the greater the differences between the snow depth fractiles calculated in various distributions. As a rule, the more the distribution upper tail tends asymptotically more smoothly towards zero, the greater the values of fractiles defined with the same period $\bar{T}$.

For the mean return periods of more than 3o years, the fractiles calculated in the Fréchet distribution are greater than the ones in the Gumbel distribution, while the latter are greater than the ones calculated in lognormal distribution or Pearson type III, etc.

Usually there are only subiective reasons for preferring one to the other of the above mentioned distributions. The Gumbel distribution seems to be more adequate for the advantages of certain mathematical connexions and continuities in the analysis of safety against snow load. In this case, in terms of mean $m_1$ and coefficient of variation $V_1$ of annual maxima of snow depth on the ground, the fractiles $x_p$ of snow depth defined by probability p of having smaller values than $x_p$ in N years are calculated by the formula:

$$x_p = m_1\left\{ 1+ \left[\left(\frac{-\ln\ln\frac{1}{p}}{1,282} - o,45\right) + \frac{\ln N}{1,282}\right] V_1 \right\}$$

respectively:

$$x_p = m_1(1+K_N V_1) \qquad (2)$$

where the values of $K_N$ in terms of p and N are the following:

| N, years \ 1-p | 0.10 | 0.05 | 0.02 | 0.01 |
|---|---|---|---|---|
| 1 | 1.304 | 1.966 | 2.593 | 3.138 |
| 2 | 1.846 | 2.400 | 3.134 | 3.670 |
| 5 | 2.560 | 3.122 | 3.849 | 4.394 |
| 10 | 3.101 | 3.662 | 4.389 | 4.964 |
| 20 | 3.641 | 4.202 | 4.705 | 5.473 |
| 30 | 3.958 | 4.519 | 5.246 | 5.790 |
| 50 | 4.356 | 4.917 | 5.644 | 6.290 |
| 100 | 4.896 | 5.457 | 6.184 | 6.728 |

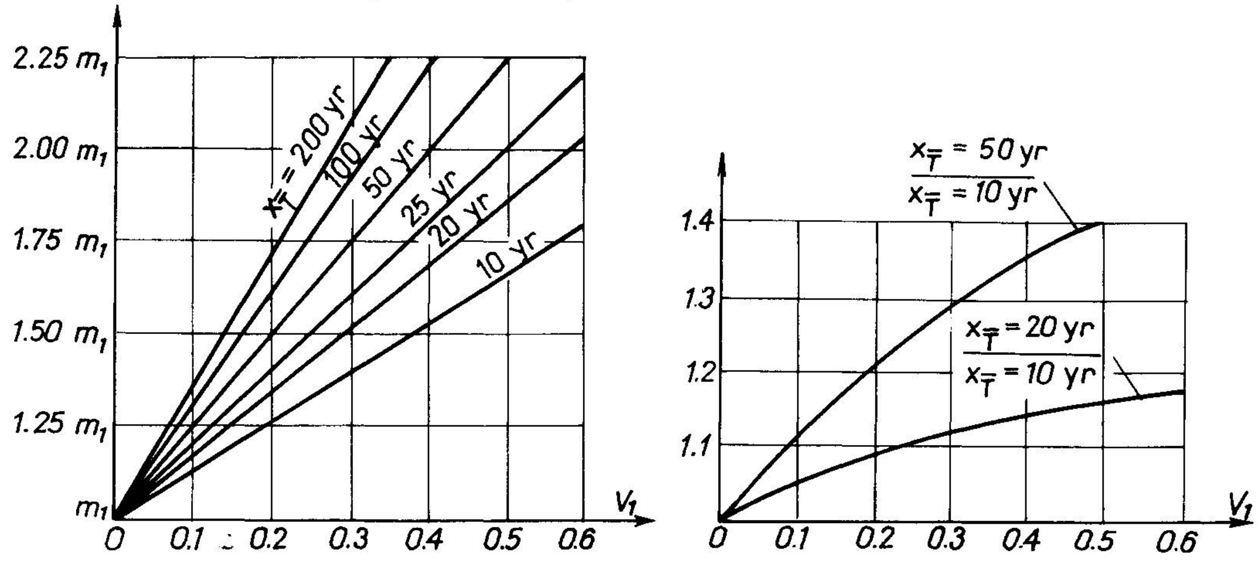
In order to use formula (2) and for the analysis of safety of the roof structure against snow load on a second moment format, it is necessary that meteorological information should be shown by means of two basic maps:

(i) the map of the mean of annual maxima of the snow depth, $m_1$;
(ii) the map of coefficient of variation of annual maxima of the snow depth, $V_1$.

With them, any values of load fractiles can be calculated directly in different sites by formula (2). The coefficient of variation $V_1$ and the mean $m_1$ are thus the basic indicators of climate severity of an area. It is to be noted that the values of $V_1$ can be very high, for example in Romania they are frequently higher than value 0.45 estimatively codified by J.C.S.S. [2].

The effect of snow depth coefficient of variation on snow depth fractiles is shown in fig.1.

As a joined action of both wind and snow the result of snow depth measurements greatly depends on local topographical conditions and built environment in the vecinity of the area where the meteorological study is made.




a. b.

*Fig.1 The fractiles of snow load as function of the coefficient of variation of annual maxima of snow depth.*

For example, the following parallel values of snow depth on the ground have been noted at two meteorological stations in Bucharest, located respectively in the north-out of town and in the south-within the town, on a small elevation:

| | Bucharest Baneasa(N) | Bucharest Filaret(S) |
|---|---|---|
| In January 198o | 4o cm | 52 cm |
| Mean of annual maxima | 35 | 48 |
| Absolute maximum | 1o4 | 15o |
| Annual maximum having: | | |
| $\bar{T}$ = 1o years | 65 | 92 |
| $\bar{T}$ = 5o years | 98 | 132 |

The following variations of the same snow of the 198o winter (corresponding to mean climatic conditions) are also mentioned in two different areas in Bucharest [8]:

| Zone | Snow depth | Snow load |
|---|---|---|
| (1) | 17 - 29 cm | 34 - 56 daN/m$^2$ |
| (2)a | 29 - 46 | 32 - 76 |
| b | 3o - 4o | 47 - 58 |

The effects of snowfalls blown off by the wind being strongly influenced by the local conditions, the snow depth measurements in different points of the same site are different and contain errors.

If h is annual maximum snow depth, $h_m$ the same depth measured, and $\varepsilon$ measurement error, then obviously:

$$h = h_m + \varepsilon \qquad (3)$$

$\varepsilon$ being a random variable of zero mean.

The means of h and $h_m$ are equal $m_h = m_{hm}$, while the actual snow depth coefficient of variation out to be greater than that of the measured snow depth:

$$V_h = \sqrt{V_{h_m}^2 + \frac{\sigma_\varepsilon^2}{m_h^2}} \qquad (4)$$

where $\sigma_\varepsilon^2$ is the variance of measurement error $\varepsilon$.

Therefore, when calculating snow depth fractiles with different mean return periods according to equation (2), coefficient $V_h$ should be used for $V_1$.

2. SPECIFIC GRAVITY OF SNOW

Snow load is calculated at present by the product between depth, probabilistically defined with a specified mean return period, and the snow specific gravity expressed deterministically:

(i) either by a singular numerical value;

(ii) or by deterministic function of different parameters (most frequently of depth h); for example, according to J.C.S.S. suggestion [2] :

$$(h) = 300 - 200\, e^{-1.5h}$$

It is to be noted that for depth $h > 1$ m, the specific gravity of snow $\gamma(h) > 250$ daN/m$^3$, which has also been proved by other studies and measurements [7], [1].

In accordance with this procedure, the random variability of snow load is derived exclusively from snow depth. This does not correspond to physical reality.

Snow load L, definedby the product between depth and specific gravity actually depends on both random variables, $\gamma(h)$ and h:

$$L = \gamma(h)\ h$$

The mean and coefficient of variation of load are thus calculated in terms of means and coefficients of variation of specific gravity and of depth with the relations:

$$m_L = m_{\gamma(h)}\, m_h$$
$$V_L = \sqrt{V^2_{\gamma(h)} + V^2_h} \qquad (5)$$

As the analytical expressions for $\gamma(h)$ are extremely different and reflect subjective approximations, the assessment of coefficient of variation of specific gravity should not be made analytically out of function $\gamma(h)$, but rather directly out of the measurements corresponding to various ranges of depth h.

The coefficients of variation of the mean specific gravity of snow for various ranges of depth, $V_{\gamma(h)}$ may have orders of size comparable with those of depth h, but they are likely to be smaller:

$$V_{\gamma(h)} < V_h$$

The monthly maximum specific gravities of snow in Bucharest for a period of about lo years, analysed irrespective of snow depth, are characterized by a coefficient of variation

$V_\gamma$=o,52 [11] . Obviously, the order of size of $V_\gamma$ is greater than that for $V_{\gamma(h)}$:

$$V_\gamma > V_{\gamma(h)}$$

The previous remarks show that uncertainities concerning the assessment of random snow load are much greater than those that appear out of considering as random variable only the snow depth and only in one point at one meteorological station.

## 3. SNOW BLOWN OFF BY THE WIND

The factors of changing the snow depth on the ground into the snow depth on the roof are estimated in the present codes between o.8 and o.6, depending on the wind exposure of the building. Such values generally apply to roofs with a relatively small surface placed in areas that are relatively free of obstacles, out of towns as a rule.

In towns, the snowfalls on roofs are determined by the effects of wind covering various "random" configurations of built volumes or of relief. Under such conditions it is almost impossible to select one single clear factor of passing from the depth of the snow fallen quite uniformly on the ground out of town to the snow depth on flat roofs in town.

Parallel measurements of snow depth on the ground and on the roof made in the winter of 198o in two different sites in Bucharest have shown that [8] :

(i) with roofs of limited areas and without obstacles in their vecinity or on the roof, the coefficient of blown off snow had values of about o.7 - o.8 ;

(ii) with roofs of large areas and with obstacles at their back no effect of blowing off appeared;

(iii) the snow depth measured on the ground in several various points in urban sites was in all cases different(larger or smaller) from the snow depth measured on the ground out of town at a meteorological station.

Therefore, the factors of passing from the snow depth on the ground to the snow depth on the roof seem to matter generally only for out of town sites, as for the sites in town suc such factors cannot be correctly appreciated.

As a matter of fact, for the experimental determination(on natural scale or on models) of the factors of snow accumulation on roofs of various shapes, these factors cannot be determined but with respect to snow depth on the ground near the roof. As a result, the blowing off effect cannot be separated from the accumulation effect, either in measurements or in designing.

## 4. FACTORS OF SNOW ACCUMULATION ON ROOFS

Owing to (i) the geometrical variety of structural shapes of roofs and to (ii) the conditions of exposure to the joined action of both wind and snow of various sites, the determination of snow accumulation factors can be only informative.

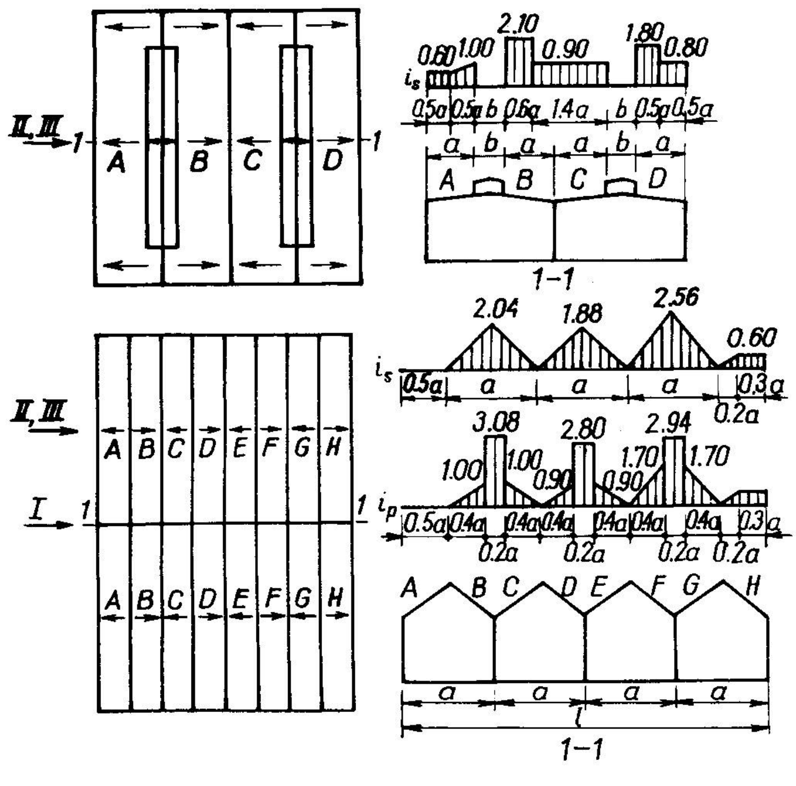
Accumulation factors can be defined:

(i) on the surface;

(ii) linear;

(iii) punctual.

The values of some of these factors, for the basic geometrical forms of roofs, are codified and verified by confronting them with the results of measurements on natural scale. The extrapolation of these results on new shapes of roofs is difficult and unreliable. As a result, it is only the modelling of snow fall in wind tunnel, that can provide designing guiding lines. Such results for different shapes of roofs are shown in [6] and are illustrated in fig.2.




a. Roof with skylight

b.Folded plate type roof

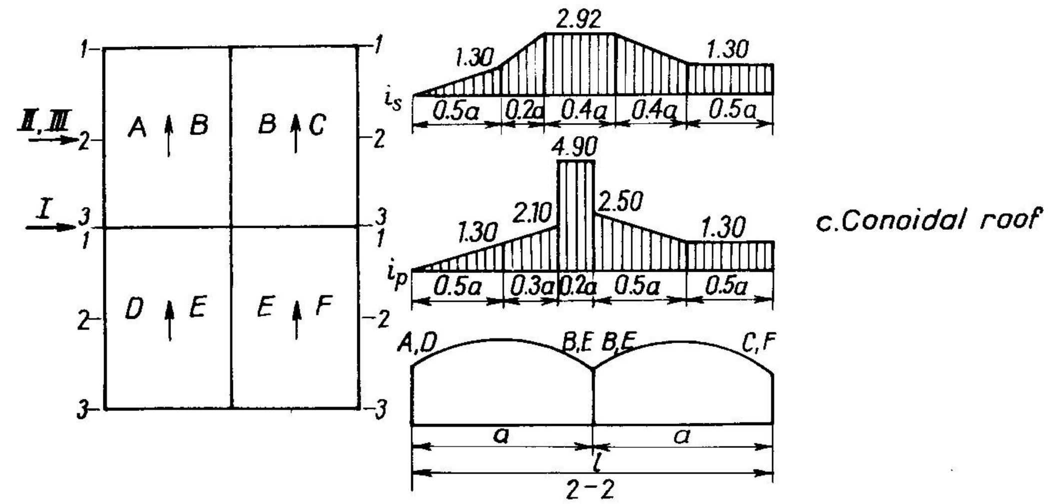



*Fig.2 Within test snow accumulation coefficients*

It must be noted that generally, the values of accumulation factors in the Soviet loading code are coupled in the calculation of snow loading with snow depths defined with small return periods(2-5 yr.), while the Canadian and American building codes use larger return periods (3o-1oo yr.). The maximum values of codified accumulation factors are usually $\leq 3$, fig.3; however, parallel measurements of snow depth on the ground and on the roof in the conditions of roofs with moderate subsidence behind some obstacles (skylight) were amplified on the roof up to 4-5 times the mean depth measured on the ground [8].

As a rule, the maxima of structural effects of snow load are achieved on an asymmetrical load scheme of the roof,thus expressing the wind effect. The responsability of the choice of accumulated snow load scheme on the roof for the designing of the structure and of its parts belongs entirely to the designer and cannot,therefore,be transfered to codes and code-producing committees.

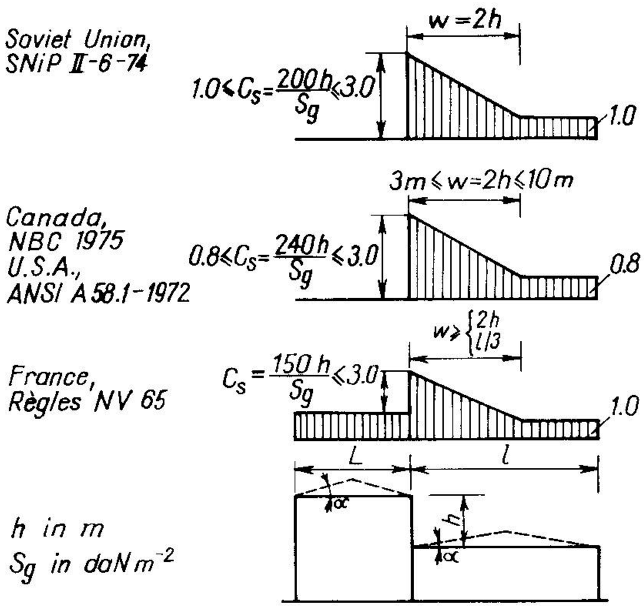



*Fig.3 Snow accumulation coefficients for the lower level of multilevel roofs.*

## 5. THE SAFETY OF STRUCTURES WITH SNOW LOAD

To analyse the safety of the roof structure with snow load and not to determine this load is the ultimate purpose in the activity of a civil engineer.
Including the meteorological information in terms of probability, the analysis of the safety of metal or reinforced concrete buildings can be made on a second moment format.
Considerind the annual maxima of snow load L, Gumbel distributed for maxima and characterized by mean $m_1$ and coefficient of variation $V_1$, the maxima in N years of load, $L_N$ are also Gumbel distributed and characterized by the mean:

$$m_{L_N} = m_L \left[1+0{,}78(\ln N)V_1\right] > m_1 \qquad (6)$$

and the coefficient of variation:

$$V_{L_N} = \frac{V_1}{1+0{,}78(\ln N)V_1} < V_1 \qquad (7)$$

Let $S_N$ be the sectional (or unitary) effect of permanent and snow loads on a member of the roof structure ,expressed in terms of them by a linear relationship:

$$S_N = a\,D + b\,L_N$$

where the deterministic factors a and b depend on the geometry of the structure, calculation method, etc.

The mean and coefficient of variation of loads effect $S_N$ are obvionsly:

$$m_{S_N} = a\,m_D + b\,m_{L_N} \qquad (8)$$

$$V_{S_N} = \frac{\sqrt{a^2 m_D^2 V_D^2 + b^2 m_{L_N}^2 V_{L_N}^2}}{m_D + m_{L_N}} \qquad (9)$$

where $m_D$ and $V_D$ are the mean and coefficient of variation of permanent load D.

Let R be the sectional (or unitary) random strength opposed to $S_N$ having mean $m_R$ and coefficient of variation $V_R$ depending on basic strength of material M by an usually linear or quasi linear relationship:

$$R = c\,M$$

respectively:

$$m_R = c\,m_M \qquad (10)$$
$$V_R \cong V_M$$

For the further calculation schematizing the distributions of random variables R and S as being of an asymmetrical lognormal type (positive asymmetry), the dimensioning relationship is the following [3]:

$$C_{necessary} = \frac{m_R \text{ necessary}}{m_{S_N}} = e^{\beta\sqrt{V_{S_N}^2+V_R^2}} \qquad (11)$$

and that of checking:

$$\beta_{effective} = \frac{\ln \frac{m_R}{m_{S_N}}}{\sqrt{V_{S_N}^2+V_R^2}} \qquad (12)$$

The correspondences between the reliability index $\beta$ and the probability of failure of the roof during N years $P_{f,N}$ are the known ones:

| $P_{f,N}$ | $10^{-2}$ | $10^{-3}$ | $10^{-4}$ | $10^{-5}$ | $10^{-6}$ | $10^{-7}$ |
|---|---|---|---|---|---|---|
| $\beta$ | 2,32 | 3,09 | 3.72 | 4.27 | 4.75 | 5.20 |

etc.

By means of relations (11) and (12), the results of the dimensioning and checking based on probability can be compared to the results of the analysis according to a deterministic or semiprobabilistic procedures. Generally it can be estimated that:

(i) The sectional areas provided by probabilistic calculation appear to be greater than those obtained by applying the codified procedure; with low reliability of the order of $P_f = 10^{-3}$ the differences are only of about 20%, but with probabilities $P_f \cong 10^{-5}$ such differences come up to over 50%;

(ii) It can be appreciated that for light roofs where snow load represents more than half of the total load of the roof, to apply the codified procedure may lead to smaller reliabilities than those accepted by standards for other types of structures and loads;

(iii) In order to prevent such situations, many specifications use additional overload factors for lightweight roofs. These factors are defined in terms of the ratio between the snow load and the total load of roofs (snow included) and generally have values ranging from 1.0 to 1.3; these values are not selected on probabilistic bases and consequently the degree of safety cannot be strictly appreciated;

(iv) With roofs where snow load has the main weight in dimensioning the structure, the use of calibration of the semiprobabilistic design on a second moment format is a necessary way of improving the current procedures of designing.

## 6. CONCLUSIONS

1. Snow depth in a given site depend on the climatic severity of the site, as well as on the local topographical conditions and the built environment in the neighbourhood of the area where the measurements are made.

2. The engineer's uncertainities concerning the assessment of random snow load depend not only of the random character of stratum depth but also on the random character, for a given depth, of the specific gravity of snow.

3. In determining experimentally, both on models and on natural scale, of roof snow accumulation factors, the effect of blown off snow cannot be separated from the accumulation effect.

4. For determining the factors of snow accumulation on roofs of special shapes, the extrapolation of load schemes from known shapes is unreliable.

5. The use of certain additional deterministic safety factors for designing lightweight roofs does not give a realistic image of their safety, which can de quantized only based on probability.

## NOTATIONS

$C$ central safety factor
$D$ dead load
$h$ snow depth on ground
$K_N$ factor for calculating snow depth fractiles
$L$ snow load
$m$ mean of a random variable
$N$ structure life, years
$P_{f,N}$ probability of failure during N years
$R$ sectional or unitary strength

$\underline{S}$ sectional or unitary load effect
T mean return period of snow depth
V coefficient of variation of a random variable
$x_p$ snow depth fractile
$\beta$ safety index
$\gamma$ specific gravity of snow

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I

# The influence of earthquake forces on the selection of structural form

Influence des forces séismiques sur le choix du système et de la forme d'une structure

Der Einfluss von Erdbebenlasten auf die Wahl des Systems und der Form eines Tragwerks

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## SUMMARY
In this century, structural systems for multistory buildings have been developed which can successfully resist severe earthquakes. The current empirical seismic design approach seems to assure life safety, but the attainment of the stated performance criteria for the various levels of earthquake intensity is only vaguely secured. New procedures using inelastic dynamic analysis make it possible to design structural configurations which can control the magnitude and locations of inelastic deformations and internal earthquake forces. These new procedures give the designer practical tools to modify seismic reponse to achieve economical solutions for any degree of damage control by regulating the relative strength between beams and columns.

## RESUME
Au 20e siècle, des systèmes structuraux ont été appliqués avec succès à des bâtiments élévés résistants aux tremblements de terre. Les principes empiriques actuels de dimensionnement semblent protéger la vie humaine, mais les critères de performance ne sont pas remplis de façon certaine pour différentes intensités séismiques. De nouvelles méthodes d'analyse dynamique inélastique rendent possible le dimensionnement structural, et permettent de localiser les dérformations inélastiques et de déterminer l'intensité des tensions dues aux forces séismiques. Ces nouvelles méthodes permettent au projeteur de déterminer le comportement séismique de la structure pour différentes intensités; les dommages peuvent être limités par modification de la résistance relative des poutres et des colonnes. Des solutions économiques peuvent être ainsi réalisées.

## ZUSAMMENFASSUNG
In diesem Jahrhundert wurden statische Systeme für Hochhäuser entwickelt, die auch sehr starken Erdbeben erfolgreich Widerstand leisten können. Die heutigen empirischen seismischen Bemessungsgrundsätze scheinen die Sicherheit von Leben zu garantieren, die Erfüllung der dargestellten Ausführungskriterien für verschiedene Erdbebenintensitäten ist jedoch nur vage gewährleistet. Neue Verfahren mit inelastischen dynamischen Analysen ermöglichen den Entwurf statischer Anordnungen, die die Kontrolle der Grösse und des Ortes inelastischer Deformationen und innerer Erdbebenkräfte erlauben. Die neuen Verfahren dienen dem Ingenieur als praktisches Werkzeug zur Anpassung des seismischen Verhaltens zwecks Erreichen wirtschaftlicher Lösungen für jeden Grad der Schadenkontrolle, indem die relative Festigkeit zwischen den Balken und Stützen angepasst wird.

## INTRODUCTION

Almost a million deaths, in this century alone, have been caused by earthquakes--and most of these deaths were in damaged or collapsed buildings. Obviously, protection of life must be the primary objective of earthquake engineering of structures. The great Kanto, Japan, earthquake of 1923, in which about 70,000 people lost their lives, spurred the initiation of specific consideration of seismic resistance in the design of structures.

In the past, only certain regions of the globe were considered earthquake-prone areas, but with the development of highly sensitive seismographs, and with increasing population density in most of the world, the extent and number of such regions have been considerably enlarged.

In the initial period of consideration of seismic resistance, lateral forces were considered to be a percentage of the weight of the structure (1 to 2%); in later developments, forces similar in magnitude to those required for wind were applied. The structural solutions for earthquake resistance paralleled those developed for wind resistance, since in both cases, resistance of buildings to lateral loads were being considered.

Although random earthquake effects have components in all directions, engineering consideration has traditionally been given only to the horizontal resistance of structures. Resistance in the vertical direction has been largely neglected, since buildings have inherent capacity to resist a substantial increase in vertical loads.

It is essential to stress that there is a major difference between wind and earthquake loads. Wind loads are externally applied pressures of air moving around the building; earthquake loads, on the other hand, are inertia forces generated in the building as a response to motions of the ground upon which it rests. Despite this fundamental difference, earthquake forces have traditionally been treated as externally applied loads, since familiar procedures were available for analysis.

### Stone and wood structures

Over the centuries, the determination of structural form has been defined by the construction materials used. Looking back through history at the monumental structures (not considering local, native-type housing), we find that stone was the primary construction material used in most of the world, although in certain areas many monumental structures were also constructed of wood. Stone structures used arches to bridge space, while timber structures used wooden beams to span reasonably long spaces. (Fig. 1)

The earthquake resistance of the two structural materials differs substantially. Heavy stone buildings are very rigid and have significant strength, with a substantial tolerance for overloads. The two basic mechanisms to withstand earthquakes are the shear resistance, which provides extreme rigidity, and the resistance to overturning, which utilizes the reserve compression capacity. Such structures ride back and forth with the moving ground. (Fig. 2a) The gravity load stress component of heavy stone buildings (i.e., the Egyptian pyramids, medieval castles and cathedrals) leaves a substantial margin for seismic overstress, and therefore, these short-period structures possess a relatively good degree of seismic resistance. Even many slender minarets have survived strong earthquakes.

Wood structures on the other hand, are incomparably lighter, and consequently develop substantially lower seismic inertia forces. Their flexible columns and beams bend during an earthquake and, thus, flexure is the basic mode of resistance (Fig. 2b). Also, their joints permit a degree of relative deformation between the connecting columns and beams. The flexibility of the columns and beams, with their comparatively higher strength-to-weight ratio, and the pliability of the joints, make wood a very good structural material for seismic resistance. Many larger timber structures have survived severe earthquakes.

## Contemporary skeleton structures

Towards the end of the last century, with the introduction of skeleton buildings, our modern structural forms for multistory buildings began to be developed. The first such building, the 10-story Home Insurance Building in Chicago, was built in 1883 and had cast iron columns and I-beams. These initial developments of high rise structures, before the turn of the century, occurred not only in cities like Chicago, and New York (where there is no earthquake risk be considered), but also in other cities where earthquakes had long been recognized as an important factor. San Francisco had a number of these "tall" skeleton buildings clad with stone or masonry which survived the 1906 earthquake relatively well.

It must be kept in mind, however, that most of the resistance to lateral forces in these buildings was derived from the stone and masonry cladding, and from the interior masonry partitions; the skeletons in these buildings barely participated in resisting the lateral forces. In many earthquakes, the actual earthquake resistance of such heavily clad buildings proved to be substantial, and more nearly represented the old stone construction than the evolving skeletal-structural type.

In modern high rise buildings, glass and light curtain wall exterior envelopes and lightweight interior partitions, are gradually replacing the heavy masonry cladding and heavy interior partitions previously used. This evolution effected a change in the actual lateral resistance of our multistory structures--the skeleton is now becoming the primary element of lateral resistance, no longer assisted by cladding or partitions. Consequently, new design philosophies for lateral resistance are evolving.

While skeletons of the first generation of heavily clad buildings designed to a wind drift limit of 1/330 performed well under wind effects, the newer, bare-bones buildings, designed to a wind drift limit of 1/500 to 1/600, occasionally exhibit serious serviceability deficiencies, not the least of which is occupant discomfort.

## Ductile Moment Resisting Space Frame

In the period between World Wars I and II, while the rigid frame was being adopted as the primary structural system for steel and concrete high rise buildings, new developments were simultaneously occurring in seismic engineering. Progress in the theory of structural dynamics shed new light on the response forces of structural systems. It became apparent that the forces developed in an elastic structure responding to a moderately intense earthquake are several times (4 to 6) higher than those specified by the codes. On the other hand, observations of the behavior of contemporary structures in earthquakes showed that, despite the relatively low design forces, these structures performed reasonably well. To reconcile the large gap between such theoretical and actual design force levels, the conclusion was drawn that structures undergo inelastic deformations during earthquake response; when the structure begins to yield (i.e., deform inelastically), no further increase of inertia forces occurs. Thus, the actual strength which is provided in the structure determines when yielding will begin and how far the structure will deform into the inelastic range by mobilizing its ductility. A structure of greater strength will delay the onset of yielding, and will utilize less ductility. Conversely, a structure designed and built with lower strength, will start yielding sooner and will deform much farther into the inelastic range, demanding a higher ductility. (Fig. 3).

Theoretical studies on single-degree-of-freedom systems indicate that a displacement ductility of 4 - 6 is needed for structures designed for code forces to resist an El Centro 1940-type earthquake. A new earthquake design approach thus resulted, in which a balance between strength and inelastic deformability (ductility) was designed into the structure. Structures composed of brittle materials (having little inherent ductility) are designed for high forces (brittle box-type structures for a system coefficient of K = 1.33), while structures composed of ductile columns and beams are designed for substantially lower forces--e.g., K = 0.67. Thus, in the 1950's the prevalent rigid frame was equipped with ductility in all its columns, beams, and their connections, and the resulting ductile moment-resisting space frame (DMRSF) became the principal earthquake resistant structural form for high rise buildings of steel and concrete. To prevent eventual instability, caused by column sidesway mechanisms resulting from hinging at column ends, and to assure the desirable beam sidesway mechanism (Fig. 4), a set of rules was incorporated into codes requiring that in a given direction the moment capacity of columns be larger than that of the beams at a joint. The intention was that hinging should occur in the beams and not in the columns.

An important advantage perceived from using the flexible moment resisting space frame was the longer period of vibration, as compared with that of a more rigid structure, which results in lower earthquake forces.

## Flexibility - ductility

It is important to draw a distinction between the terms flexibility, as opposed to rigidity, and ductility, as opposed to brittleness. (Fig. 5) These terms are sometimes mistakenly interchanged. Beyond the limit of its elastic deformations, a flexible element or structure can be either brittle (susceptible to crushing or disintegration) or ductile (pliable). On the other hand, a ductile structure can be either rigid or flexible, depending on the amount of lateral deflection caused by a unit of lateral force. Inelastic deformations can occur only in ductile elements or structures.

Once inelastic deformability (ductility) became identified as the main structural characteristic needed to resist earthquakes, it followed that the strength of all the members of the structure should be governed by flexure, and that no premature brittle shear failures should interfere with the process of yielding at critical nodes of the structure. Ductile yielding in critical nodes redistributes the moments to the lesser stressed locations of the structure until all hinges needed to create a collapse mechanism are developed.

A set of details to assure ductility of columns and beams and their connections was introduced into earthquake codes (Figs. 6 and 7). At the same time, the use of brittle elements such as shear walls for the primary elements of seismic resistance of concrete buildings was discouraged by specifying higher seismic forces for them. In a similar way, braced frames (vertical trusses) were discouraged, to avoid compressive buckling failures of steel truss members.

It should be noted that for earthquake resistance of inelastic buildings, deformations are as important as stresses, since beyond the elastic limit of member capacities, plastic deformations continue to increase with stresses remaining at yield level. Therefore, any approach considering only elastic stresses cannot deal realistically with either safety or damage control.

## Choice of structural material

The two basic materials for highrise structures are steel and concrete, while wood continues to be used effectively for lowrise buildings. Due to its inherent ductility as a material, structural steel is often perceived to be superior for seismic resistance. Experimental studies during the 70s demonstrated that reinforced concrete beams, columns and shear walls can be specially detailed to have ductility in a range similar to that of structural steel members. On the other hand, some steel sections may buckle before reaching their yield capacity, thus having no ductility at all.

Seismic resistance of a structure depends not so much upon the structural material used as upon the structural form selected, and the details used. It also depends upon the appropriateness of the structural form for the given material. For example, steel is more efficiently used for linear members (columns and beams), whereas concrete is more effective in plate action such as in slabs and shear walls. It is possible to design good and bad structures with either of the two materials.

## Nonstructural elements

During the last quarter of a century, while the concept of the ductile moment resisting space frame has been predominant for earthquake resistance, there has been an ongoing transition in the construction industry in the use of "nonstructural" materials. The older, heavier cladding materials such as stone and solid masonry are gradually being replaced in building elevations with glass and light curtain walls; masonry partitions are being replaced with lightweight partitions made of gypsum, wood or plastic. In some countries outside the United States, hollow clay tiles are replacing solid masonry in exterior walls and in interior partitions. Some of the newer partition materials are rigid and brittle (hollow clay tiles), while others create resilient partition assemblies. Most of the newer materials have low strength, consequently, their contribution to the lateral resistance of the frame may be minimal.

Observations in earthquakes, and inelastic dynamic studies, indicate that modern buildings may have interstory distortions in the range of 1 to 1-1/2 in. in response to severe earthquakes.

When flexible, resilient, interior partitions and exterior cladding panels are used with flexible frames, the seismic distortions of the frame can be followed by the nonstructural elements without damage. However, when brittle infill is incorporated into flexible frames, the seismic distortions of the frame can cause damage to the brittle partitions, occasionally with an explosive release of energy and damage to the frame. Therefore, brittle elements should not be built into flexible skeletons, unless details are provided to allow frame distortion without straining and damaging the rigid, brittle nonstructural elements. (Fig. 8)

## Experience in earthquakes

Ever since the San Francisco earthquake of 1906, evidence is available that a great number of contemporary buildings in various areas of the earth have been subjected to and have withstood severe earthquakes. As construction and design procedures have improved, records show that an increasing percentage of multistory buildings have performed satisfactorily, amply demonstrating that it is possible to build structures which will withstand earthquakes of major intensity. While brittle-type structures, such as those of unreinforced masonry, usually have performed poorly in major 'quakes (as might have been expected), structures designed with some consideration for earthquake forces have demonstrated a full spectrum of behavior ranging from poor through merely adequate to excellent. In numerous cases (e.g., in Central America, Romania) where recent codes were implemented, the performance of some structures has been very good, confirming the general validity of the direction of our codes for regular well proportioned buildings without drastic stiffness changes from level to level.

On the other hand, if we look at the examples of failure of contemporary concrete and steel buildings in the various 'quakes, we can see that most of the failed structures were not designed for earthquake resistance. Of the failures of buildings designed in accordance with recent codes many were directly attributable to drastic changes in stiffness between successive stories. The extreme changes caused large distortions of the more flexible stories, with subsequent collapse due to brittleness of columns.

Following each of the recent earthquakes, modifications in either the design forces or reinforcement details, or both, were implemented into codes, thus generally advancing the state-of-the-art.

It should be noted that a significant number of the buildings observed in earthquakes contained "nonstructural" elements which were not considered in the analysis, but which substantially contributed to their seismic resistance. Therefore, caution must be exercised in drawing conclusions concerning the adequacy of structural systems based solely on the observation of performance of such buildings.

Experience in earthquakes shows that while details are extremely important, the role of structural form in determining seismic response cannot be overemphasized. No amount of excellent details can improve the poor performance of an ill-conceived structural system. Sources of major distress during earthquakes have been structural layout deficiencies such as: substantial asymmetry of members resisting the lateral forces; large differences in plan dimensions between the two orthogonal directions; large discontinuities in stiffness and strength between subsequent levels, and others.

## Shear walls and braced frames as elements for lateral resistance

Concrete load bearing walls (unreinforced and reinforced) have been incorporated into buildings for as long as concrete has been used. In the 1950s modern shear walls, acting as cantilevers, were introduced to stiffen frame-type buildings. Owing to the high rigidity of walls, as compared to the frame, the entire lateral resistance was assigned to the walls, and the columns, slabs and beams were designed for gravity loads only. While this approach may appear safe, since all the loads and their effects seem to be accounted for, the frame is actually underdesigned, because its deflection-induced moments, shears, and axial forces have not been considered.

In the 1960s, practical analytical methods were developed to consider the interaction between frames and shear walls. The major beneficial effect of the interaction is a set of internal forces (tension and compression, shown in Fig. 9) between the frame and the walls which drastically reduces the overall deflection, thus increasing the stiffness of the interactive system. Such added stiffness, without added cost, permits the construction of very tall buildings economically, mostly without paying a premium for height. This means that such buildings are designed for gravity loads, and the effects of wind are accommodated within the 33% increase in allowable gravity load stresses.

These advanced analytical methods have resulted in new shear wall-frame configurations which could not have been devised previously due to a lack of analytical tools. New structural types have dramatically improved the efficiency of wind-resisting structures.

While new rational approaches have been utilized to develop alternative structural systems for wind resistance, the continued use of elastic analysis (which inadequately represents inelastic response) has inhibited the development of more effective and efficient structural configurations for seismic resistance.

## Observation of seismic performance of shear walls

Beginning with the Kanto (Japan) earthquake of 1923, the earthquake records of recent times indicate that buildings containing shear walls perform considerably better than frame-type buildings, both with respect to safety against collapse as well as control of damage to nonstructural elements. The presence of shear walls, even when substantially cracked during an earthquake, prevents collapse by hindering formation of column sidesway mechanisms. The presence of the shear walls also limits the interstory distortions, thus lessening damage to nonstructural elements.

On the contrary, frame structures are much more flexible, and respond with substantial interstory distortions during an earthquake, leading to damage of brittle nonstructural elements, finishes and contents of the building as shown in Fig. 10 from the 1967 Caracas, Venezuela earthquake.

While in earthquake after earthquake examples can be cited in which shear wall structures have demonstrated superior behavior, as compared with frame structures, codes still discourage the use of shear walls by specifying higher load factors for them and lower permissible strength. It was only during the early '70s that questions were raised regarding the preferential status of the ductile moment resisting space frame versus shear walls.

While in the early 1980s there is already widespread recognition of the need for shear walls to improve seismic resistance of concrete structures, the discussion continues among professionals on the choice of flexible vs. rigid structures. To avoid resonance, it is necessary to stay away from a certain period of vibration, when the period of the soil is definitely known. However, except for Mexico City, there are very few locations in which the period is known with certainty--too few to warrant making an exact period the major structural criterion. In addition, period determination is not yet an accurate exercise; a taller, rigid structure may have the same period as a flexible structure with half the number of stories. The many advantageous aspects of more rigid shear wall and truss-type buildings such as: superior damage control due to smaller interstory drift, simplified ductility details, simplicity of detailing of nonstructural elements, etc. by far outweigh the possible lower inertia response forces generated in flexible structures.

## Development of new structural forms -- shear walls

In the '70s, extensive experimental programs on shear walls were carried out at a number of universities; these studies produced a large body of information showing that shear walls can be made ductile by special proportioning and detailing of reinforcement. Walls with a balance betweeen flexural and shear strength, with an upper limit on shear stresses, and with proper reinforcement distribution and detailing can provide the ductility level that may be required in major earthquakes. It may not be desirable to rely on ductility of shear walls as the primary energy-dissipating mechanism of a structure; however, available ductility in walls makes them a most obvious element to interact with frames for lateral bracing and to provide a second line of defense for energy dissipation.

## Slitted shear walls

Another method for utilizing ductility in shear walls was recently developed by Japanese engineers. The slitted wall concept (Fig. 11) was introduced to make possible the incorporation of shear wall panels into steel frame multistory buildings. Slitting the walls converts a story-high panel into a number of flexible vertical elements, achieving two objectives: (a) the brittle shear-governed behavior of the story-high shear panel is transformed into flexure-governed behavior of the individual vertical slender elements; and (b) the high stiffness of the wall panel is reduced so that the deformation of the slitted wall becomes close to that of the frame. As a result, the two elements can cooperate effectively in resisting lateral loads.

## Shear wall-frame interactive systems

The state-of-the-art in seismic design for highrise buildings is still dominated by the moment-resisting space frame. Highly efficient structural systems for wind resistance, developed on a rational basis, are only slowly and cautiously being introduced in high risk seismic regions. For the low and moderate risk seismic regions, new work is in progress to provide earthquake resistance by using the efficient shear wall-frame interactive structural systems designed for wind as a point of departure. The strength and detailing of the beams is then modified so that they will respond inelastically and dissipate energy. The walls and columns are designed to remain elastic for the design earthquake, and to utilize a limited amount of their inelasticity only in the event of a hypothetical maximum credible earthquake. Such desired response can only be achieved by using inelastic analysis techniques.

## Inelastic response history

Recent improvements in computer technology (both hardware and software) have brought inelastic dynamic response history analysis within the reach of practical design. Response history analysis is a step-by-step tracing of the response of a structure to an earthquake accelerogram in small time increments. The use of such analysis makes it possible to consider the yielding of individual members and to incorporate into the structure the necessary ductility details only where required, thus eliminating the costly ductility details in places where they cannot be utilized. The new approach leads to a more rational design resulting in controlled seismic behavior. It also provides the engineer with a new, powerful, tool with which he can devise innovative, more effective structural configurations to dissipate seismic energy--systems we were not able to devise before because of a lack of means to analyze them. It is hoped that the inelastic procedure will advance earthquake resistant structural design technology just as the shear wall-frame interactive methodology advanced wind resistant structural design in the 1960s.

## Serviceability and stiffness

The major structural difference between wind and earthquake-resisting structures is that for wind, the members resist factored loads within their elastic range, below the yield level, while for earthquakes, the members are designed to deform beyond the yield level, into the inelastic range.

Inelastic deformations of structural members cause their permanent distortions. Although such distortions of beams may create unsightly cracking, it is the accumulation and spread of inelastic deformations in columns which may eventually endanger the structural stability. To ensure life safety, therefore, inelastic deformations in columns must be kept within tolerable limits.

Serviceability as related to tall, wind-resisting structures, is determined by interstory drift which affects nonstructural elements, and by wind vibrations which affect comfort of occupants.

In the evolution of modern structural systems for tall, wind-resisting structures, stiffness of the overall system has been the primary criterion for acceptance, measured by interstory drift. The maximum allowable drift, in turn, depends on the ability of nonstructural elements to distort without distress and without losing their ability to function as needed. This, of course, is in addition to stability considerations.

In seismic structures, comfort of occupants during an earthquake becomes of secondary importance, while interstory drift and its effect on nonstructural elements becomes the prime serviceability consideration.

It has been established that the magnitude of overall deformations in a structure are about the same, whether it resists an earthquake elastically or inelastically. Since earthquake damage to nonstructural elements in a structure is largely determined by the interstory distortions, the control of such damage requires that a sufficient amount of initial elastic stiffness be designed into the structure. Depending on the intensity of the "design" earthquake, the minimum rigidity needed for acceptable control of seismic damage to nonstructural elements may exceed that required to control wind drift.

The relationships between stiffness and strength of a structure, and its ductility demand, are complex. More rigid structures generate higher seismic inertia forces and consequently, require a higher design strength which affects ductility demands. In general the major influence on ductility demand results from the actual strength level for which a structure has been designed.

The selection of the structural system for seismic resistance can be made in two stages:

1. Stiffness is selected to result in a wind drift within the elastic range, to satisfy the wind serviceability requirements (drift and vibration);

2. Then, the strength level determined for wind resistance is gradually modified to result in an elastic plus inelastic (ductile) seismic drift within a limit to assure damage control.

It follows that a desirable overall approach to finding an economical and efficient seismic structure is to start with the most efficient structural configuration based on stiffness to satisfy wind requirements, and then progressively modify the strength of its members under earthquake forces until a desirable balance between strength and ductility is achieved.

## CONCLUSION:

Looking towards the next stage of earthquake engineering development, it seems that our ability to protect life should be taken for granted, and the task of protecting property through damage control should now receive our full attention. Flexible frames, detailed for ductility, are well suited to provide earthquake resistance in structures (such as bridges, stadiums, some types of industrial buildings, etc.) where large earthquake distortions do not damage nonstructural elements and do not affect their subsequent functional performance. However, in residential and commercial buildings, in which the structure represents only 20-25% of the building's cost, controlling damage through control of earthquake deformations becomes an important design consideration. For such buildings, structural systems containing shear walls and trusses are gradually gaining acceptance in seismic regions. For wider utilization of shear walls and trusses, designed on a rational basis for damage control and for controlled inelastic seismic action, new simple inelastic analysis methods (static or dynamic) now need to be developed.

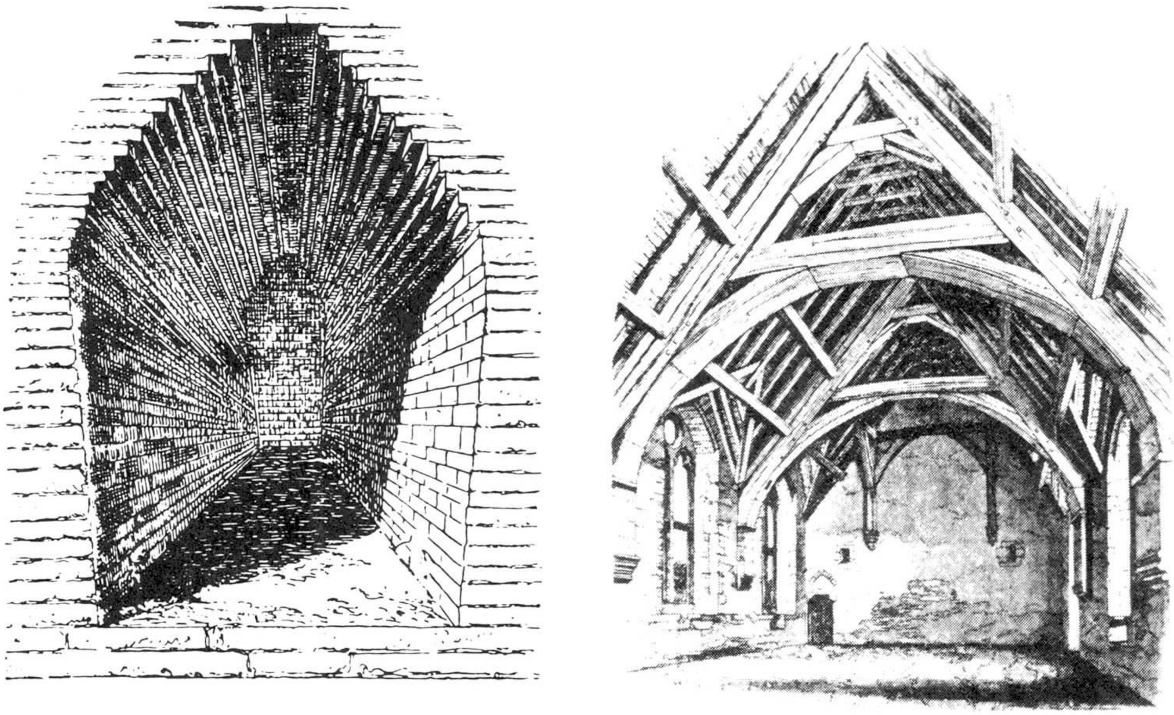

Fig. 1: Stone arches, timber structures

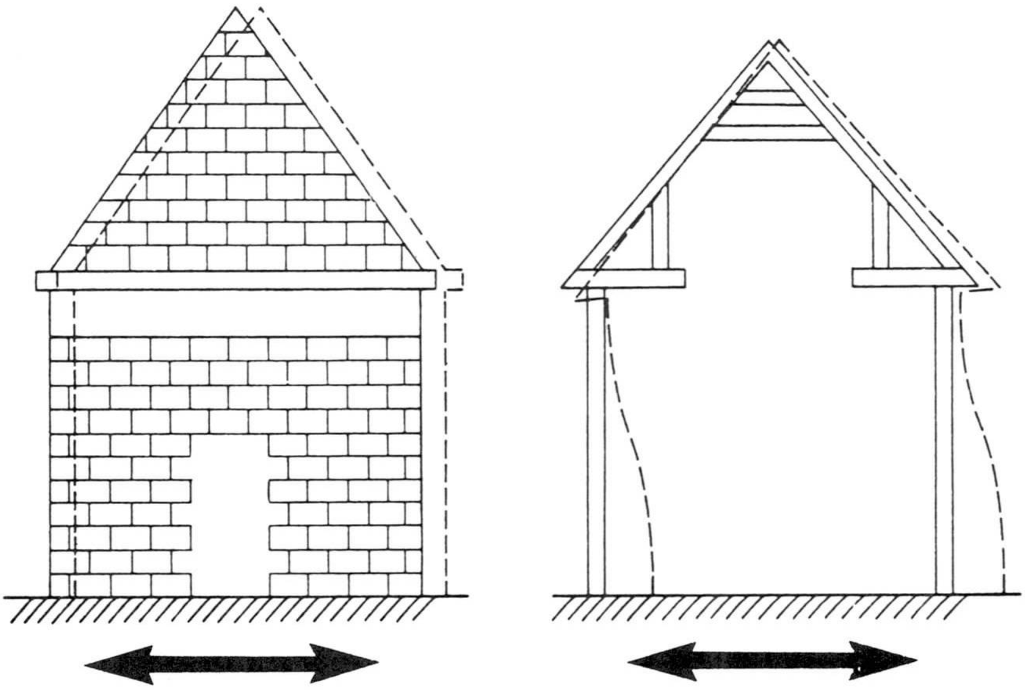

Fig. 2: Earthquake response of stone and timber structures

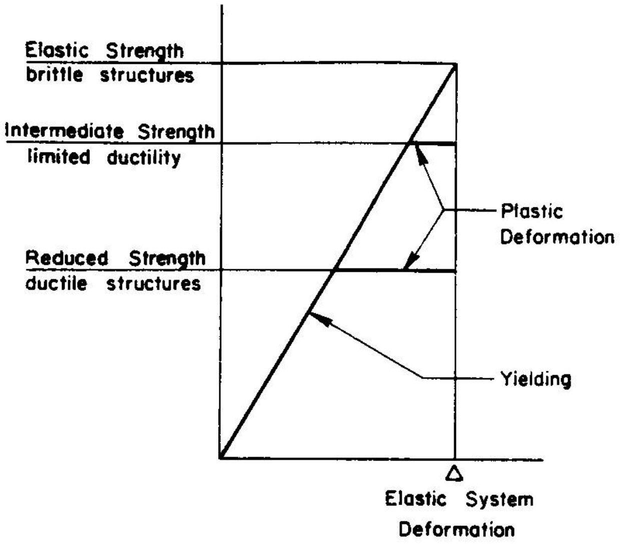


Fig. 3: Strength vs. deformation in elastic and inelastic structures

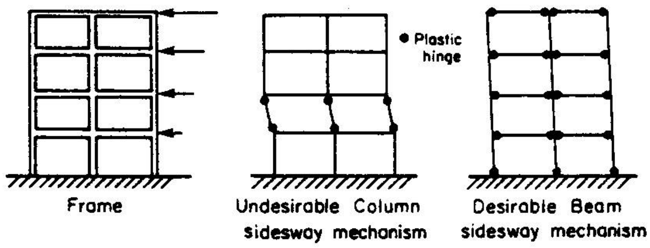


Fig. 4: Possible mechanisms due to formation of plastic hinges in frames subjected to lateral loads.

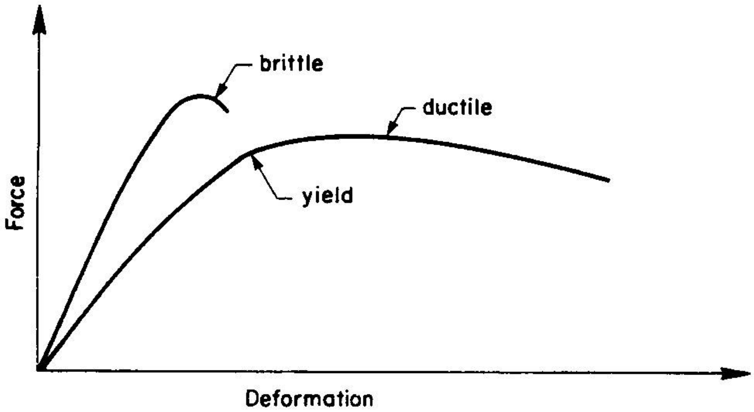


Fig. 5: Brittle and ductile failure mechanisms

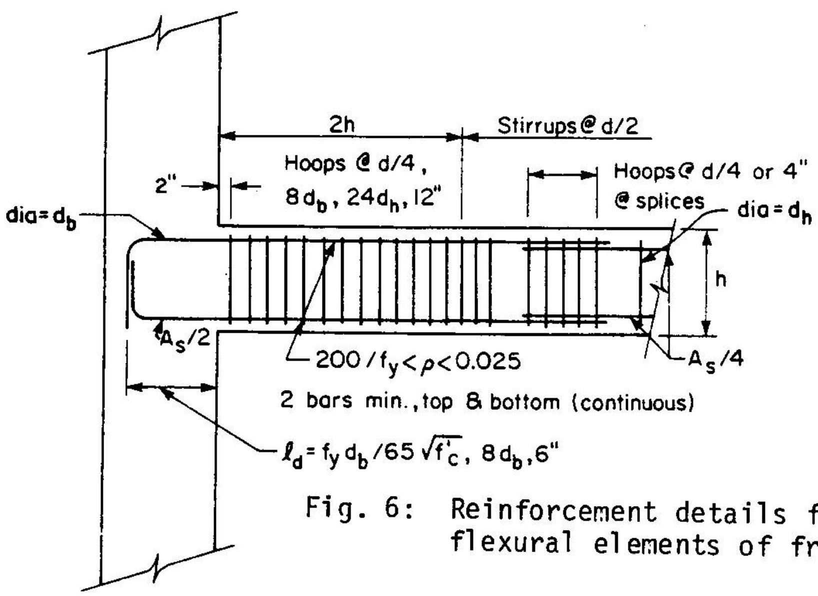


Fig. 6: Reinforcement details for flexural elements of frames

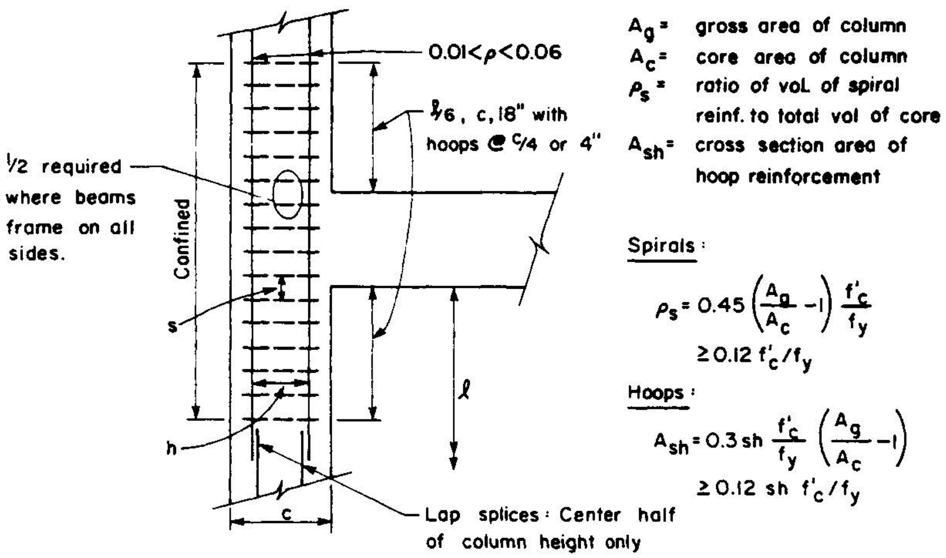


Fig. 7: Reinforcement details for columns of frames

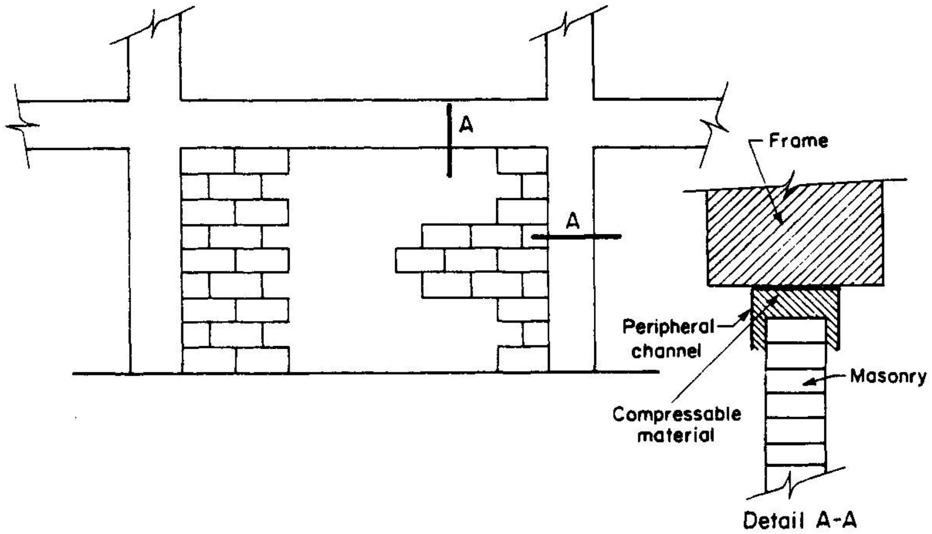


Fig. 8: Connecting rigid partitions into flexible frames

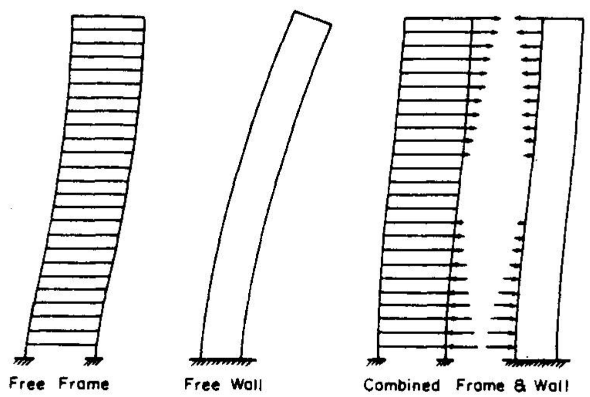


Fig. 9: Shear wall-frame interaction


Fig. 10: Damage to nonstructural elements in 1967 Caracas, Venezuela earthquake

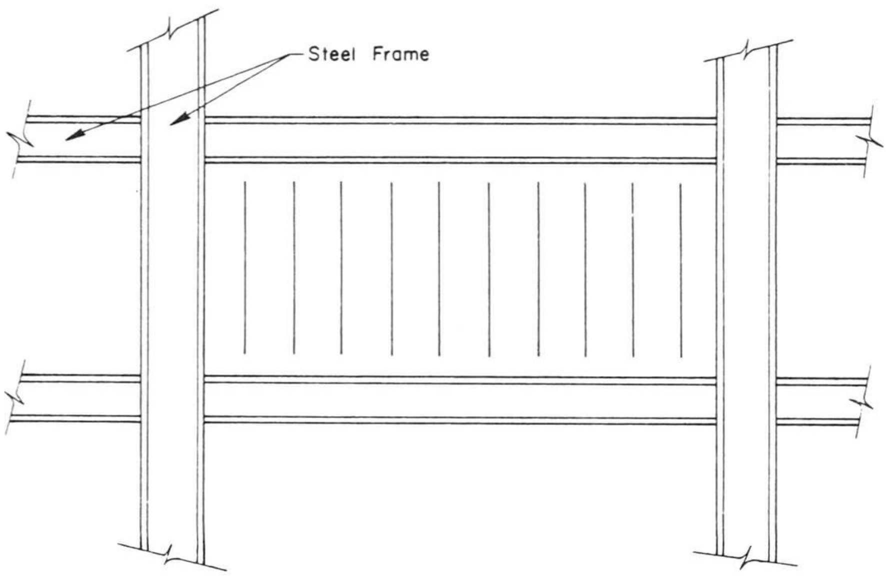


Fig. 11: Slitted walls

I

# The influence of hydraulic forces on the selection of structural form

L'influence des forces hydrauliques sur le choix du système et de la forme d'une structure

Einfluss hydromechanischer Kräfte auf die Wahl des Systems und der Form eines Tragwerks

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## SUMMARY
Besides that the hydraulic forces have an influence on the structure, the shape of the structure may often influence the hydraulic loads as well. A distinction should be made between hydrostatic forces which are for instance caused by a difference in head between two water levels and dynamic forces as e.g. loads resulting from the energy in waves. We give some examples of the structures which are mainly subject to hydrostatic loading. Recommendations have been discussed in order to reduce wave forces and are followed by examples.

## RESUME
Les forces hydrauliques agissent sur une structure. Mais la forme d'une structure peut aussi influencer ces forces hydrauliques. Les forces hydrostatiques peuvent être causées, par exemple, par une différence de hauteur de deux niveaux d'eau et les forces dynamiques peuvent être dues à l'énergie de la houle. Des exemples de structures surtout soumises à des charges statiques sont présentés. Des recommendations sont proposées afin de réduire les forces dues à la houle et sont illustrées par quelques exemples.

## ZUSAMMENFASSUNG
Hydromechanische Kräfte beeinflussen ein Tragwerk, aber durch die Form eines Tragwerks können diese hydromechanischen Kräfte beeinflusst werden. Man unterscheidet zwischen hydrostatischen Kräften, die z.B. durch zwei unterschiedlich hohe Wasserspiegel entstehen, und hydrodynamischen Kräften, die durch Wellen verursacht werden. Als Beispiele werden Tragwerke besprochen, die hauptsächlich durch hydrostatische Kräfte beansprucht werden. Es werden ausserdem Empfehlungen zur Reduktion der hydrodynamischen Kräfte gegeben und einige Beispiele dazu präsentiert.

## 1. INTRODUCTION

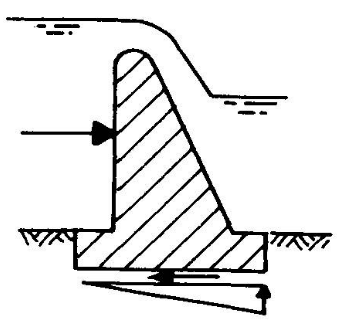
The designer of hydraulic structures is confronted with a large variety of loads originating from different sources. It is his task to give the structure such a shape, so that these loads are obviously transferred to the subsoil and in an economical way. A complication is that the shape may influence the hydraulic loads and vice versa, which can be an advantage. This is an impediment to a more systematic treatment of the influence of hydraulic forces in selecting structural shapes.
For this reason the paper will deal with the subject as follows.
First of all a survey will be given of the hydraulic forces involved, making a distinction between static and dynamic forces. A few examples of structures which are mainly subject to static loading will be described.
Then some topics on wave action will be discussed and some general recommendations to reduce wave loads on structures will be given, followed by a few examples.
Finally wave power installations will be described as an example of the reverse principle, viz. not the dissipation but the accumulation of wave energy.

## 2. HYDRAULIC FORCES

Hydraulic forces can be distinguished in static and dynamic loads.
To the hydrostatic loads belong:
a. A difference in head as is e.g. the case with a weir, which is made to separate two different water levels. Differences in head may also arise due to long waves such as surges and translation waves.
b. All-sided water pressures which act on submerged structures as e.g. a sub-aqueous tunnel.
c. Flow induced stationnary forces (drag forces on bridge piers etc.).
Dynamic loads can be divided into:
a. Standing wave loads resulting from in time and depth changing pressures on a structure (storm surge barriers etc.).
b. Wave forces consisting of inertia and drag forces (acting on legs of offshore platforms etc.).
c. Wave impacts caused by waves that strike the surfaces of the structures (storm surge barriers, breakwaters etc.).
It should be noted that the first two loads have a semi-static character, whereas the last one is really dynamic, happening within parts of a second.

## 3. STRUCTURES LOADED BY HYDROSTATIC FORCE

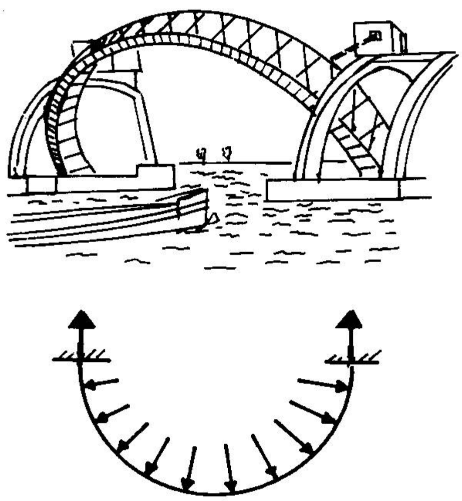
a. Difference in head



FIXED WEIR

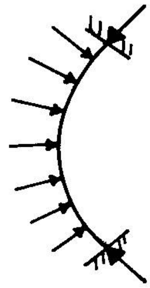
The external hydrostatic forces develop internal stresses in the structure which transmits the external load together with the dead weight to the subsoil. The shape of a fixed weir e.g. is defined by a vertical wall with a certain width at the bottom to transmit the bending moments, shear and normal forces to the footing and by a width of the floorslab in accordance with the bearing capacity of the subsoil (both horizontally and vertically). If the hydraulic loads are large and the external horizontal stability is not ensured this can be ascertained by increasing the dimensions of the structure (increased dead weight and friction).

The shape of a tension curve in steel or a pressure curve in concrete gives the possibility to optimise the use of the construction material, where a uniform load occurs due to e.g. hydraulic head. This principle for instance has been applied at the semi-circular gates of the movable weirs in the Lower Rhine [1]. A complication is the connection of the steel gate to the concrete structure. The free outflow of the undershot must consequently be possible near the piers and abutments, otherwise the blockage of flow would built up pressures, which endanger the equal load distribution on which the gate design has been based.



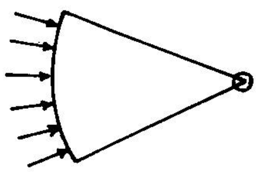
MOVABLEWEIR

If the pressure head is very large as e.g. in a deep valley, a concrete arch dam can carry the hydraulic force horizontally to the subsoil. It has the shape of a pressure curve and spans between the side slopes of the valley.



ARCH DAM

The circular shape can also been seen in a certain type of lock-gate as the sector gate with vertical axis. Due to the circular shape and the fact that the waterpressure always acts perpendicular to the surface, the resulting horizontal load runs through the rotation axis.
Therefore, the forces on the operational equipment are lower and also the required power when the gate must be opened under a head or closed during a flow, than in the case of e.g. mitre gates.
The rising sector gate, which is more used and sometimes called segment gate or taintor gate, has the same advantages as the above mentioned sector gate.
In order to avoid vibrations as much as possible during the raising of the gates it is important to place the rotation axis exactly in the centre of the steel plating.



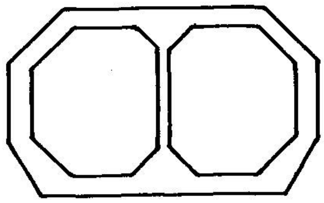
SECTOR GATE

Above a few examples have been described of how to reduce the construction material or the required power of the operational equipment by applying circular shapes. It should be stressed however, that this shape is generally more labour consuming than a simple flat structure, or occupies a large space. For that reason flat structures often lead to more economical solutions, especially in countries where the labour cost are high.

b. All-sided water pressure

The circular cross-section proofs to be the best shape in case of all-sided water pressure as e.g. for tunnels which are crossing waterways and for piping under the influence of ground water, as this gives minimum bending

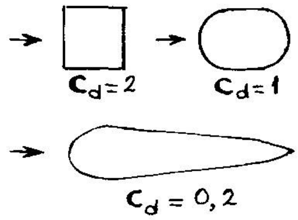
moments.
Where this principle is abandoned for other reasons, it sometimes shows its influence in the cross-section as is shown in the figure of the 2-track metro tunnel in Rotterdam (immersed type built in concrete). Although the curved shape is preferable from the load bearing point of view, it is often abandoned in immersed tunnel design because it can lead to superfluous space around the generally rectangular traffic gauge (free width x free height). Therefore this can lead to a greater length of the tunnel, because the road surface or the rail tracks are situated at a greater depth below the river bottom than in a rectangular structure [2]. This results in higher cost and that's why most immersed tunnels in the Netherlands and in many other countries have a rectangular shape.



RAILWAY-TUNNEL

c. Forces by constant flow

To minimise flow induced forces as e.g. on a bridge pier in a river it is preferable to round the edges of the structure.
In this way the drag force acting on the pier can be reduced considerably as is shown in the figure




$F = C_d \, \frac{1}{2} \rho \, A \, u^2$ in which

$F$ = drag force (N)

$C_d$ = drag coefficient (-)

$\rho$ = density of water (kg/m$^3$)

$A$ = pier area perpendicular to the direction of the flow (m$^2$)

$u$ = undisturbed flow velocity (m/s)

Generally the drag force is small compared with the other loads acting on the pier, but the streamlined shape has a favourable effect on the flow as well, increasing the discharge coefficient of the remaining cross sectional area of the river.
It should be added that the flow may cause a force perpendicular to the drag force as well, viz. the lift force which can have an alternating character.

## 4. CHARACTERISATION OF A WAVE-FIELD AND THE TRANSFER TO DESIGN LOADS

The wave loads acting on a structure can be described as follows:

$$S_w(f) = O^2(f) \times S_\eta(f)$$

in which

$S_w(f)$ = spectral density of wave loads (load spectrum)

$O(f)$ = transfer function; the waves are assumed to be long crested and perpendicular to the structure

$S_\eta(f)$ = spectral density of incoming waves (wave spectrum)

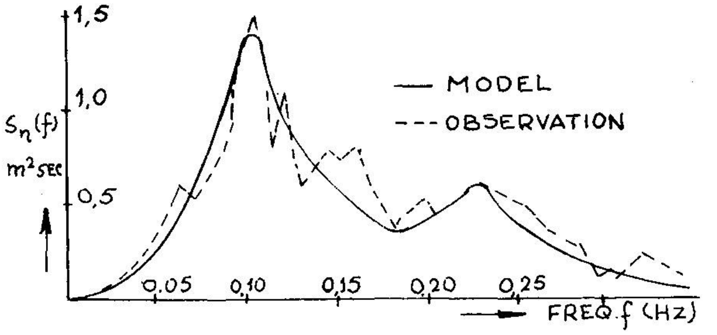
$f$ = frequency

It is obvious that the shape of the wave spectrum and the transfer

function determines the shape of the load spectrum. Figure A gives an example of a wave spectrum in the sea delta in the south of the Netherlands. The wave spectrum is double peaked, which is caused by two energy sources [3].
- a low frequency wave energy peak resulting from the open sea windfield propagating over the shoals and
- a high frequency wave energy peak generated by local wind over the shoals.

In figure B the transfer function is given, that has been derived from calculations and was tested in laboratory wave flumes.
In this case the transfer function has high values in that frequency range where the highest peak in the wave spectrum occurs. This peak is in the low frequency range, that means the low frequency wave energy is mainly responsible for the load on the Eastern Scheldt storm surge barrier [4].

FIG.A WAVE SPECTRUM IN FRONT OF THE BARRIER



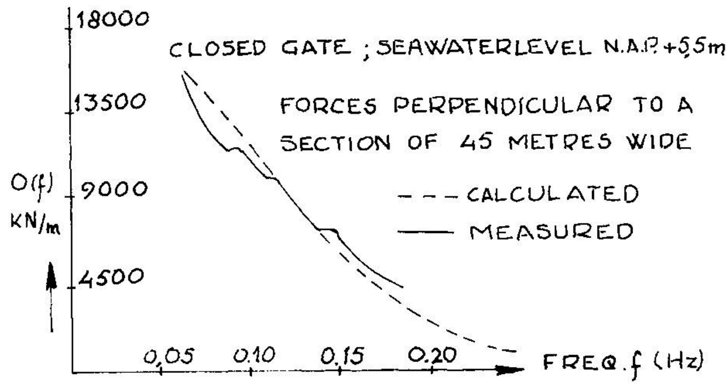



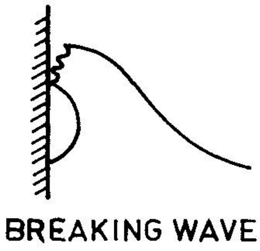

FIG. B TRANSFER FUNCTION

This barrier which is now under construction, consists of 63 openings divided by piers. The openings are 39,5 m wide and will be closed by vertical lift gates during storms.
Normally the gates will be open in order not to interfere with the salt water- and tidal regime in the Eastern Scheldt basin. The worst loading for the barrier will be during the maximum storm surge level. The difference in head between sea level and basin level and the amount of low frequency wave energy penetrating from the North Sea across the shoals are then both at a maximum.

## 5. RECOMMENDATIONS WITH RESPECT TO THE DYNAMIC LOADS BY WAVE FORCES

The difference in shape of a structure, mainly loaded by a static hydraulic force and one mainly loaded by a dynamic hydraulic force will now be discussed. In the case of a static load the purpose is to find an appropriate shape to bear the given load. When the wave load dominates there is a moving mass of water containing a quantity of energy. The problem is to find a solution to let pass as much energy as allowed and to dissipate the rest-energy as much as possible with a minimum of resulting forces. It may seem a strange solution to let pass a certain amount of energy, in other words to accept that a part of the waves and thus a quantity of water runs over the structure. But it should be noted that many structures like storm surge barriers are located near the coast line. A large water area remains behind, viz. the river or estuary which has been closed off. A limited amount of water overtopping the gates will

not raise the water level too much. When water is stopped abruptly an impact is introduced. This happens when a free water surface (exposed to the air externally or internally in a bubble or a cavitation under vapour pressure) touches a fixed surface parallel with the free water level. When for instance a wave is stemmed in a blind corner of the structure, the moving mass is stopped abruptly and the water decelerates briskly. The result of this is a large impact on the surface. Such a wave impact can be several times larger than the statical load by the same head. Blind corners should therefore be avoided as much as possible.
A good solution can be to design a structure with a low top level to have waves roll over under extreme conditions. Or the opposite; a high bottom level that cannot be reached by the top of the waves (e.g. off-shore platform).
Other solutions are: to have the water escape, to give the fixed wall an other inclination or to incorporate an elastic buffer.



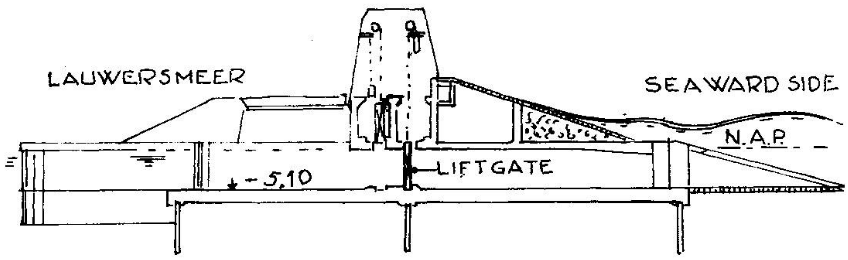
BREAKING WAVE

If possible one should prevent that waves break just in front of the surface, which gives the dominant effect of a hammershock (although somewhat damped by enclosed air). It should be noted that if a structural shape is suitable to limit the discharge as little as possible, it will also minimise the loads onto the structure, because the hydraulic force that acts on the structure is equal to the force which acts from the structure to the water.

## 6. STRUCTURES MAINLY LOADED BY THE DYNAMIC HYDRAULIC FORCES

### a. Discharge sluices at the Lauwersmeer and Grevelingen

If an discharge sluice has a rectangular cross-section the waves can hit the roof, causing a large impact. A solution is to design the roof high enough above the water level.
Another approach has been followed with the discharge sluices of the Lauwersmeer and the Grevelingen, which are bothe in the Netherlands.
A low roof has been designed and the gentle seaward side slope of the adjacent dam is continued on the top of the roof of the sluice. In stormy weather the water level is above the roof, and the waves cannot enter the sluice but are attenuated on the slope. The wave load on the gate situated half way the sluice tube is small then. In normal weather conditions, during ebb-tide smaller waves do enter the sluice.



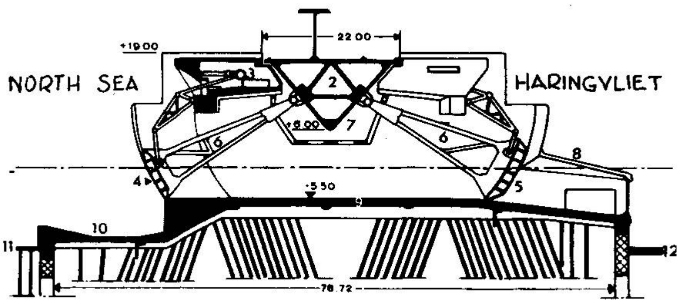

To reduce wave impact on the gate, a groove in the roof near the gate permits the watermass to escape in vertical direction, like in a surge tank preventing water hammer in a long pipe.

As mentioned before the Eastern Scheldt storm surge barrier will be provided with vertical lift gates.
The plating is located on the basin side, while the horizontal girders which transmit the loads to the piers protrude on the seaside. Closed girders would be submitted to vertical wave impact. To reduce this impact

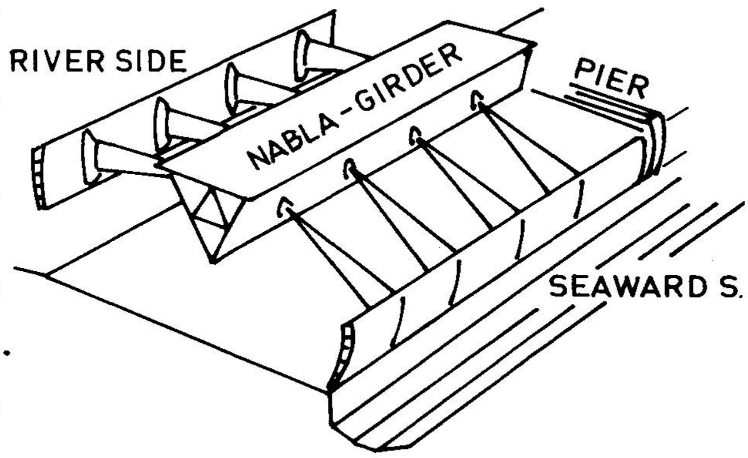
as much as possible the girders will be designed as an open framework constructed of tubular steel work.

b. Discharge sluices of the Haringvliet

The discharge sluices at the Haringvliet in the south west of the Netherlands [5] are another example of a structure mainly loaded by dynamic hydraulic forces. The structure has seventeen 56 m wide apertures, each of which can be closed by a double set of rising sector gates which are connected to a large bridge girder. The girder that spans from pier to pier transmits the static and dynamic forces on the gates to the piers. The gates can be swung in open or closed position by the long steel arm-connections between gates and girder.

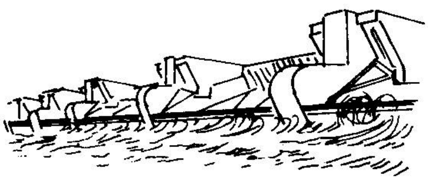



The cross-section of the 'Nabla girder' is a triangle with one horizontal side on top and one vertex on the bottom. The sharp bottom edge prevents the impact of wave tops on a flat horizontal surface, which would give rise to large upward forces on the girder.
The seaward side of the sixteen piers are flush with the steel gates, to prevent blind corners.
The gates on the seaward side are lower than the gates on the river side. So during a storm surge, part of the waves run over the gates on the seaward side, thus lowering the dynamic force on the gates under these extreme circumstances.

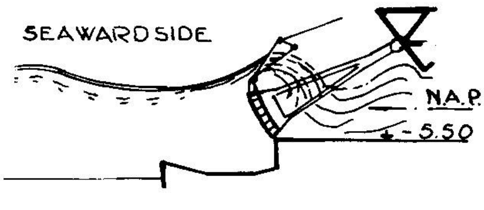


HARINGVLIET-OUTLET SLUICES

The backward slope of the gates on the river side is more favourable regarding the wave impacts than the forward slope of the gates on the seaward side.



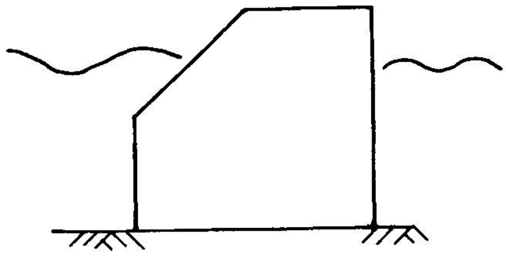
The floorslab in front of the outer gates is designed at a low level to prevent waves from breaking just before the gates, which would cause very large forces.
In the horizontal plane the abutments form a blind corner with the outer steel gates. In order to avoid high loads on the gate, the wall of the abutment is provided with a wave absorption chamber. This chamber with a few reinforced concrete columns is situated where the abutment meets the outer gate.



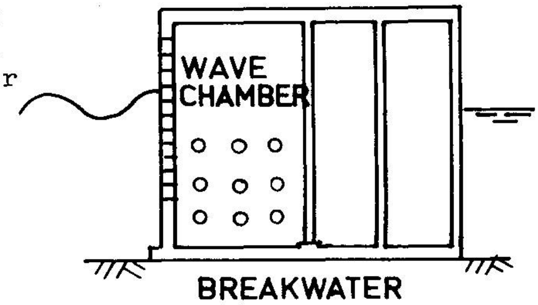

The abutments connect the sluices to the dam which has gently sloping faces against which the waves can dissipate gradually by rolling out, like on a natural beach.

c. Breakwaters

Breakwaters are built to protect harbours and sluices etc. against wave attack and are sometimes used to train currents.
Also here it is often tried to make the top as low as is allowed by its protectional function in order to reduce wave loads acting on it.
If the breakwater is constructed of caissons, loads can be reduced by providing the seaward face with an inclined plane which tends to make the waves roll over.

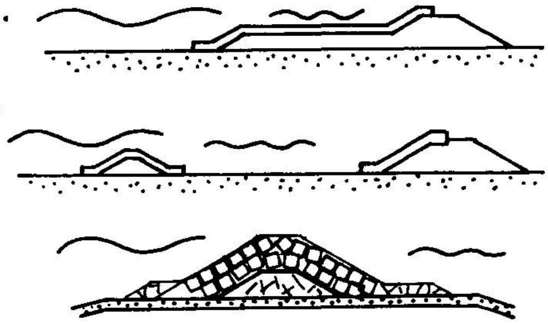

BREAKWATER

Moreover a stabilising effect results from the vertical component of the wave induced loads. In some designs (e.g. the breakwater berth in the port of Be-Como, Canada) the caisson is provided with a perforated forepart. The way this reduces wave loads will be briefly explained under d (offshore structures).



BREAKWATER

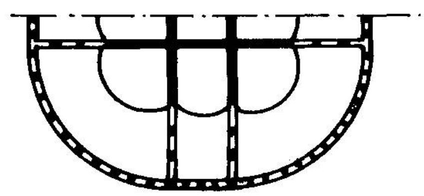
To break large waves, before they strike the breakwater, an underwater berm or dam can be made in front of the breakwater. An alternative for the caisson dam is the open structure built with gravel, rubble and often covered with concrete blocks [6] Such a mole has a gentle slope and a large porosity of about 50% in which the energy is dissipated.
Due to the low reflection coefficient, the water level variation has already been reduced.


MOLE

d. Offshore structures

Ekofisk

In some types of offshore structures, like Ekofisk (a storage tank for crude oil with an outer diameter of 92 m and a height of 90 m, placed in the North Sea at a water depth of about 70 m), perforated outer walls are used. The impact of a wave is related to the problem of the so-called added watermass which suddenly has to be decelerated. The quantity of this added watermass is greatly reduced by perforating the wall.

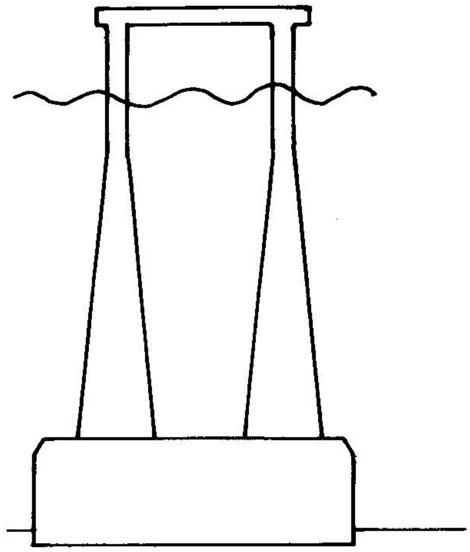

PERFORATED WALL

Sometimes more than two walls are used, the openings in each successive wall being smaller than in the preceding one, to dissipate energy in successive steps.

Andoc [7]

The wave loads influence the shape of the structure to be designed. Another example is found in the design of offshore platforms, where e.g. relatively small circular sections of the legs near the sea level reduce the wave forces. The top deck of this structure has been designed

to be outside the influence of the waves. To reduce as much as possible the wave loads caused by orbital motion the diameter of the legs should be small.
Requirements are often contradictory and may arise from other functions (stability during transport, installation of pipes in the interior etc.). It is the designers task to find the right compromise.

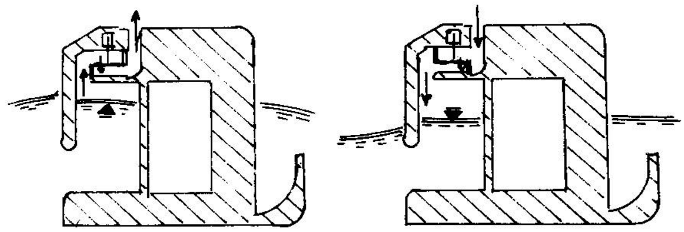


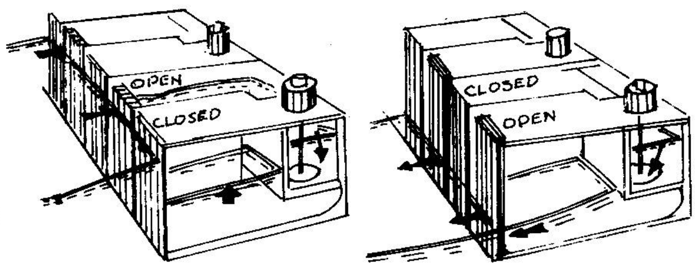
OFFSHORE STRUCTURE

The same principle is used in floating structures. The large floating body of the structure is below the water level further beyond the main reach of the dynamic action of the waves. The top deck of the platform is fixed with legs on the floating body.

e. Wave power installations

There are structures with installations to accumulate wave energy instead of dissipating the wave energy as described before. Some are designed as floating devices in order to convert wave energy into electrical energy. One type has the shape of a container without a bottom and a hole in the top. As the water level inside the box oscillates by wave action, air is forced through the hole and can be made to drive a turbine.
A ship-type craft is moored in the sea of Japan [8] .
Another project is designed as rectangular boxes installed on the seabed. The incoming wave enters an upper reservoir. Water is forced through turbines to lower reservoirs and rushes out as the wave retreats.





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I

# The influence of man-made loads on selection of structural form

Influence des charges d'origine humaine sur le choix du système et la forme d'une structure

Einfluss der auf den Menschen zurückzuführenden Einwirkungen auf die Wahl der Tragwerksform

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**SUMMARY**
In the choice of structural forms, man-made loads often have a rather limited place. Their influence mainly depends on their variability and concentration in space

**RESUME**
Dans le choix des formes structurales, les charges d'origine humaine ne tiennent souvent qu'une place assez modeste. Leur influence dépend principalement de leur liberté et leur concentration dans l'espace.

**ZUSAMMENFASSUNG**
Bei der Wahl der Tragwerksform finden die auf den Menschen zurückzuführenden Einwirkungen meist wenig Beachtung. Ihr Einfluss hängt vorwiegend von ihren Beweglichkeiten und der Konzentration im Raum ab.

## 1. INTRODUCTION

Man-made loads (m.m.l.) are rarely given in technical articles as a justification of the selection of one structural form. Yet the erection of a pedestrian bridge, road bridge and railway bridge performed on the basis of identical constructional data (excluding loads) would result into variations, usually notable and sometimes very wide, not only in structure size or constructional details, but also regarding the structural form.

## 2. SIGNIFICATION OF "MAN-MADE LOAD" (m.m.l.)

2.1 Referring to modern concepts, the meaning of the word "load" is extended here and covers the words "action" (i.e. imposed forces or deformations) and "situation" (including such phenomena as fire, impact, partial destruction).

2.2 The term "man-made" is to be understood as contrary to the word "natural" (natural actions have been dealt with in prior introductory reports); the reciprocal influence of natural circumstances and m.m.l. is not considered here.

2.3 Human activity plays a major part in all permanent actions, such as structure and superstructure dead weight, prestressing, etc. However, most of these actions will not be taken into account further on, inasmuch as, generally, they are consequences resulting from the selected form; we shall therefore consider mainly "loads" which directly (i.e. independently of the selected design) constitute data to be taken into account in the determination of the form. They consist essentially of :

- among variable loads, the working loads which cover traffic and occupancy loads along with many others : industrial, hydraulic loads (for instance, in dams and reservoirs), etc.;

- most accidental actions, particularly the actions which result from accidents.

However, it will be shown later on that the loads resulting from the form selec tion interact with the form, either directly or as a consequence of some erection phases.

## 3. - HOW ARE m.m.l. DEPENDING ON HUMAN ACTIVITY ?

3.1 The m.m.l. dependence on human activity may be of different nature; two extreme cases are to be distinguished :

a - m.m.l. depending upon a number of human decisions, as for instance the weight of a number of vehicles or persons on a bridge or a floor;

b - m.m.l. depending upon one human decision, as is the case for instance when vehicles over a given tonnage are forbidden admittance to a park or a bridge by a guard, or when the driver of a heavy vehicle takes a forbidden route.

3.2 This discrimination leads, according to modern concepts, to different responses as regards safety against such actions, namely :

- in the first case, m.m.l. are dealt with like natural actions : design is carried out as a function of a statistical estimate of the magnitude of actions; in this case the actions are variable loads, and the applicable distribution laws are similar to those of natural actions;

- in the second case, the response involves other provisions, which may be independent of structural arrangements : for instance, assignment of responsibilities and controls; then infractions to the regulations are taken into account as accidental actions.

It can therefore be deduced that,as a function of the response to the m.m.l., their influence on the selection of structural form may be much reduced, up to zero in some cases. Thus the possibility of vehicle impact can be dealt with through a safety barrier which will, if borne by the structure, deaden and localize the force, whereas it will cancel any influence of this action on the design, if it is not borne by the structure.

3.3 However, inasmuch as a risk for the structure has been admitted, the attempts at reducing its consequences will have an influence on the selection of the structural form, which must be :

- robust (for instance, an admitted risk of partial destruction of a building by a gas explosion leads to select a form excluding "progressive collapse"),

- fit for repairing (for instance, after an impact of medium intensity).

## 4. MAN-INDUCED CONSTRAINTS, BESIDE ACTIONS, APPLIED TO PROJECTS OF STRUCTURES.

4.1 It is worth mentionning, aiming at a thorough study of the problems of selection of structural forms, which are not dealt with in any specific report, the selection of structural form is frequently conditionned, to a large extent, by constraints which cannot be identified to actions, environment or material, some of them being of human origin.

The most important are doubtless geometrical constraints.They may be mainly ascribed to environment. However, some of these constraints may proceed from traffic, as for instance in the case of a road below which a building is contemplated; such constraints may be related to m.m.l. Such is also the case of constraints resulting from the necessity of placing ducts within structures.

4.2. The rangeof constraints, induced by man, which cannot be considered as actions, is very wide. Some examples in proof thereof are given as follows.

a - Aesthetics is a purely human concern, which does not regard the choice of material only (see Session 3); it exerts a direct influence on designs, even on the selection of the forms. Such influence is the most direct where the structural shape can almost be identified with the construction shape, as in the case of bridges.

b - Noise (a consequence to human activity) and the resulting need of protection against noise (a human requirement) may lead to provide phonic screens on a bridge. Then, in a particular case near Paris, the structural form was selected so that it could include these screens; an aesthetic concern played an intermediate part in this selection.

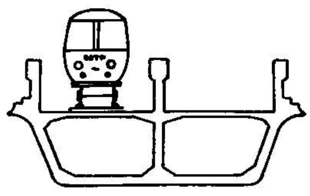


Figure 1

Likewise, in crossings for game animals above motorways, the structural form selected included equipment designed to protect the animals from the traffic light and noise.

c - Fires usually result from human activity, but they are not really loads, although they often involve high imposed deformations : the result is mainly strength losses. They may be taken into consideration in the selection of the form more or less like the other accidental forces.

These various man-made constraints will not be further studied in the following.

## 5. GENERAL PROPERTIES OF m.m.l.

5.1 These properties are related to their place in various qualitative classifications of actions, the main ones being defined in the Model Codes of International Technical Associations (notably the Volume I of J.C.S.S.). We shall therefore deal with these first.

5.1.1 We have already seen that, regarding the well-known, most usual classification of actions as permanent, variable and accidental, we shall consider variable and accidental actions mainly.

5.1.2 Actions may be taken into account as dynamic, depending on the structure : often, as regards accidental actions, rather seldom, in the case of variable actions. This may influence the form selection : some actions must compulsorily be considered dynamic (certain industrial loads), but for other actions the aim of the selection may be to consider them as quasi-static (many traffic loads) to reduce their dynamic majoration factor. Effectively, in this case, the fundamental period of the structure should be rather short; the form must therefore be selected so as to ensure a proper stiffness.

5.1.3 Almost all the m.m.l. considered here are direct actions (imposed forces). In the exceptionnal case of a m.m.l. with indirect action (imposed deformation), this feature would lead to select a form flexible enough.

5.1.4 A last general classification of Standard Codes discriminates free actions and fixed actions. In fact, this classification, though sufficient for the use of influence lines and surfaces, is very basic. It will be shown, that the structural form depends largely on this classification, which should therefore be refined. More precisely, we can discriminate :

- immutable and determined position loads,
- moving loads and/or undetermined position loads, and, in such cases, different degrees of mobility (or indetermination) :
  - . perfect linear mobility (loads on rails),
  - . approximately linear mobility (road loads),
  - . total freedom in two directions (loads on floors).

Mobility is mainly a property of loads concentrated on small areas; we shall see later on (§ 5.3 ) the problem of concentration.

5.2 Many other qualitative classifications are possible (see for instance Manual of structural safety, C.E.B Bulletin 127, pages 249 to 253). We shall indicate some other distinctions and general characteristics of m.m.l. as follows.

5.2.1 Variable m.m.l. are practically always more or less intermittent, they almost never happen to be continuously applied during the whole lifetime of structures.

5.2.2 Among variable m.m.l., some of them give rise to long duration applications, either in a single occurrence, or by cumulating several occurrences. The other ones and the accidental m.m.l. may be said short-duration m.m.l. With materials likely to creep, long-duration loads can lead to select stiff forms in order to prevent excessive deformations. Here is an example of the relationship between action and form, conditional and linked to other data.

5.2.3 It is to be noted that m.m.l. are almost always unfavourable (as they draw the structure closer to its limit states).

5.2.4 As variable m.m.l. are largely due to gravity, they are essentially directed vertically downwards *; however, some of their components may be horizontal, very important (reservoirs, silos) or secondary (traffic loads); even secondary components can exert a notable influence on the form.

Accidental m.m.l. may be horizontal, more or less freely directed.

5.2.5 A final distinction is made between loads which depend only on functionnal construction data (notably occupancy and traffic loads) and which are therefore independent of the structural form, and loads depending on the form. This is the case of water pressures in a tank : as the structural form is generally identified to the construction form, the pressures perpendicular to the walls necessarily vary with the form; in this case, the form is often selected as a function of this dependence (see § 8.2).

5.3 On the other hand, the influence of m.m.l. on the form depends largely on their concentration. An actual classification is not possible in this case; we shall show the variety of the cases with some examples :

a - for a single load, all concentrations are possible between two extreme cases :

- uniformly distributed load (actual case)

- load concentrated at one point (asymptotic case).

Moreover, the load density can also be non-uniform on the loaded area.

b - estimating the concentration is still more complex in the case of multiple loads, as it depends not only on the concentration of individual loads, but also on their number and spacing.

## 6. MODELIZATION OF LOADS.

In fact, when dealing with more or less free and more or less concentrated loads, the loads themselves are not introduced, generally, into the designs; these take into account very simplified models, which fit more accurately the above description. This is a modelization in space, and the structural form is selected with reference to such models.

* An exception is the variation in earth pressure due to variable loads applied on the backfill.

These models are selected in order to generate effects equivalent to those generated by actual loads in the structures. Actually, this aim is never perfectly reached, even though, with a view to a greater accuracy, certain m.m.l. are represented by several mutually exclusive models, taken successively into account: for instance, a model of distributed (generally uniform) loads, and a model of highly concentrated loads. Inaccuracy is generally rather small (maybe of the order of 10 %) as regards principal effects such as bending moments in critical sections; but it may reach 20 to 40 % for other effects, such as shear forces or local moments. Such inaccuracy depends in a large measure on the selected structural form; the compatibility of the model with the selected form is a factor of the form selection which, practically, is almost never considered in the selection of the form, but which should be considered a posteriori at least, in order to check the compatibility, mainly so when the eventuality of a new form is being contemplated. Sometimes, however, a loading standard gives a precision: for instance, it states that the model must be complemented by a concentrated load, in the case of a floor consisting of a slab and joists.

An exemple of the sensibility of the form to modelization is shown is the figure opposite : the modelization of the accidental impact of a boat against a bridge is limited to a force applied to its piers in the river; exclusive application of this model could logically lead to place the piers on the banks, and to bring the deck very close to water at both ends (the model did not include the application of forces to the deck, whatever the deck level above water). Obviously enough, the deck would be subjected, in case of shock, to accidental forces smaller than those applied to the pier, since they would be applied by the boat superstructures; nevertheless, it appears clearly that the form obtained by pure and simple reference to a model of the action may lead to safety problems.

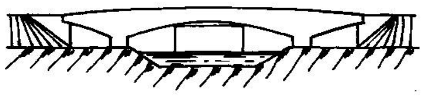


Figure 2

As the structure also is modelized for design, it must be ensured that such modelization does not introduce excessive inaccuracy. The selected form must enable to comply with this condition.

## 7. RELATIONSHIPS BETWEEN MODEL PROPERTIES AND THE SELECTION OF THE STRUCTURAL FORM.

Now we shall see, for each particular m.m.l., which general properties, according to its usual modelization, it is accredited with in the selection of the form.

Information on this, for variable then for accidental loads, is gathered in the two following tables.

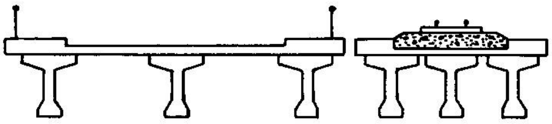
| LOADS | MODELS<br>PRINCIPAL LOADS | <br>SECONDARY LOADS | CONSEQUENCES |
|---|---|---|---|
| 1 - VARIABLE<br><br>Loads on floors (for dwellings, offices, stores, parkings, etc.)<br>Very numerous cases are to distinguish numerically according to the structure intended use : passages, stairs, archives, libraries, education, entertainment, medical attendance, etc., see the draft for ISO 2103 | Static, free loads, vertical downwards, represented by a double system of forces :<br>. uniform distributed forces (principal effects)<br>. concentrated forces (local effects) | Conventional component (representing also certain resistance requirements), with any horizontal direction, between 1 and some % of the principal loads | The double system of principal loads is qualitatively identical for almost all these loads; only the magnitude of forces varies according to the utilization; thus forms are fairly similar for all intended uses. |
| Industrial loads (for factories, sheds, wharfs, marine structures) | Same types of models as in the case of loads on floors. Besides, possible specific loads are defined individually in each particular case (heavy equipment, travelling bridge cranes); these loads are often fixed or guided, sometimes they are dynamic. | | Structural forms may differ from those of similar constructions, due to :<br>. either very different geometrical constraints,<br>. either a very different magnitude of free loads,<br>. or the existence of fixed or guided loads,<br>. and sometimes isolated supports for dynamic loads |
| Traffic loads :<br>- roads<br>- rail<br>- planes | Multiple systems of the same types as the systems of loads on floors, sometimes simplified into mixed systems (concentrated loads added to distributed loads). Sometimes the freedom of road loads is restricted, but only rail loads are currently considered as guided. | Braking components, centrifugal components, etc ..., with fixed direction | |

| | | | |
|---|---|---|---|
| - pedestrians, cycles | Single system of uniform loads; sometimes a concentrated (usually accidental) load is also contemplated. | Neglected | |
| Water pressure in dams<br><br>Liquid or gas pressure in tanks | The model is simply conformable to reality; fixed system of pressures for a given level, depending on the structural form. | Considered in exceptional cases (effect of a motion of the filling body, hydraulic gradient in an earth dam, etc.). | As the forces applied depend directly on the structural form, the form will be commonly selected so as to get forces balanced advantageously enough; |
| Pressure of stored solid materials | System of wholly or partly free pressures, which can depend on the structural form (silos). | | |
| 2 - ACCIDENTAL<br><br>Vehicle impact (car or truck, boat, plane, exceptionnally train or others) | Two cases, according to whether the forces are applied directly or not to the structure :<br>. first case : the forces are horizontal or almost horizontal (except in the case of plane impact), more or less free in position and direction,<br>. second case : forces are fixed (for instance, applied to the fastening point of a barrier) | Usually neglected | The models and their consequences depend on the selected procedure as regards safety against this possible danger. |
| Vehicle at a strictly forbidden place | Unique, highly simplified system (single vehicle), derived from the system of variable traffic loads. | Usually neglected | |
| Explosion | Special, usually complex model, which defines a partial destruction of the structure and may include uniform dynamic pressures on the non-destroyed parts. | They are due to the forces developped at the breaking points of the destroyed parts and to their fall. Often neglected despite their importance. | The model selected is conventional, with a view to selecting a robust structural form. |

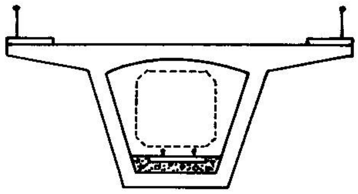
## 8. INFLUENCE OF LOADING MODELS ON THE FORM SELECTION.

It can be observed that m.m.l. as a whole, in spite of their large diversity, are introduced into the designs through a very short number of model types. Thus it is not surprising that the variety of structural forms should not be due to the variety of m.m.l. The diversity of forms is mainly derived from the other factors, notably the diversity of constraints of all natures, but these constraints are often connected to the existence or the magnitude of m.m.l.

Sometimes, for instance, its being not possible to introduce loads on areas with any disposition whatever with respect to one another, the selection of forms will be severely restricted.



instead of



A certain diversity of forms can also be brought directly by the different orders of magnitude of these loads (case of foot-bridges, road-bridges, rail-bridges, above mentioned), or even, exceptionnally, by the variation in magnitude of the modelized load, according to whether such magnitude is or not degressive as a function of the loaded area.

### 8.1 Case of variable loads, mainly vertical.

8.1.1 All models of principal loads are fairly identical : distributed loads uniform and concentrated, almost static, vertical downwards, intermittent, free.

The existence of free concentrated loads leads to design bi-dimensional structural parts (slabs, or including slabs and possibly ribs). According to the order of magnitude of the loads, to the spans, etc.,either the role of these parts is restricted to distribution, or they enact as principal girders. The heavier the loads, the more solid the forms.

8.1.2 Loads guided with accuracy seem rather exceptional; this, however, is the case of railway loads.

In this case distribution slabs are sometimes more or less completely abandoned, as in the case of certain industrial loads. In developing countries, the floor slab of certain road bridges is even sometimes performed by two lines of separated boards, though road loads are not guided accurately.

8.1.3 An intermediate case very frequent, even almost general, is the case where different natures of loads are systematically applied to distinct areas, perfectly determined. Such separation may lead to distinct bi-dimensionnal structural parts. When there is any doubt on the perennity of such separation (for instance, a cycle track may be incorporated some day into the neighbouring carriageway, even if the traffic are materially separated originally), preference is given to a common bi-dimensional part.

8.1.4 In a building where a dynamic working load is fixed (for instance, weight of a turbine), it may be advantageous in certain cases to disconnect completely its support from the building structure. This prevents the transmission of the dynamic effect to the whole structure, and retains the possibility of suppressing, through a restricted alteration, resonance with the foundation ground.

8.1.5 Following these choices, the other parts of the bearing structures are usually placed under (sometimes beside) the bi-dimensional structural parts, therefore rather freely with respect to m.m.l. It will be attempted, as far as possible, to select a structural form reducing distribution action-effects in the bi-dimensional upper parts to moderate values.

In many cases, the structure has two principal directions, and the choices of forms in longitudinal and in transverse directions may be separated, even in a single structural element.

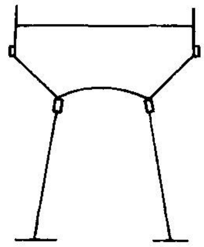
8.1.6 Secondary forces, with completely different directions, are sometimes not negligible in the selection of forms, mainly as regards supports. But they are often covered by wind, seism, accidental actions, etc., and have therefore no direct influence on the structural form.

8.2 Non-vertical pressure of solid or liquid materials.

This is mainly the case of dams, tanks and silos.

In these structures, the pressures of solid and liquid materials are distributed and applied to each point. Therefore a bi-dimensional envelope is constituted, which here again is an important part of the structures.

For the general balance of forces, the fact that they depend on the form of the envelope and are generally due to gravity leads, very often, to give the envelope and the structure itself a revolution form with vertical axis. The meridian line can usually be selected rather freely, and forces are balanced at the changes in direction through belts. Trying to get a proper balance of internal forces in various parts leads to the traditionnal mushroom-shape of tanks on top of towers.



However, in quite a number of cases (for instance, in canal bridges), functional, geometrical, or other constraints may lead to quite different forms, or even impose one form.

In the case of medium or high pressure gas tanks, the smallness of gravity forces as compared to pressures, and the pressure magnitude, lead logically to the spherical form as optimum form.

An example of a particular structure is constituted by shaft linings, which withstand occupancy or traffic loads inside, and water pressures, more particularly upwards, outside. These structures are subjected to important functional constraints. Moreover, its being impossible to provide supports as regards uplift, the upward force must be balanced through a ballast. Thus the loads are balanced locally, and action-effects are too small to exert any additional influence on the form selection.

8.3 Accidental actions.

The part they play in the form selection cannot be neglected; it is more diversified than the part played by variable actions. Their influence depends on their modelization, which depends in turn on the selected safety procedure (see above § 3.2). The selection of such procedure interacts with the selection of the form.

In all cases the degree of safety with respect to such loads is rather small; forms must therefore be selected with a view to reasonable possibilities of rehabilitation. It is sometimes advisable, with this end in view, to discard wholly welded metal structures, because the replacement of a damaged part would be very difficult, due to their form and design.

8.3.1 The treatment through structural design is analogous to the treatment of variable actions, with analogous consequences on the selection of the structural form.

- In the case of very free accidental loads, the forms are usually more solid than they would have been otherwise, but they are not, as a rule, altered fundamentally. Furthermore, it is generally thought useless to go to much expense in order to take accidental actions into account.

- In the case of more or less "fixed" accidental loads, strong structural parts must be provided at the estimated level of application of such loads.

But the other safety procedures with respect to such actions may exert very different influences upon the structural form.

8.3.2 Thus the control of these actions can exert a notable influence on the selection of a form, as shown by the following three examples :

- addition of a spillway to a dam,

- erection of the roof of a hydrocarbon tank, very close to the liquid surface, in order to prevent the accumulation of large amounts of gas, apt to explode,

- addition of equipments intended to deaden and localize impacts (barriers, guard rails, protective frames, pile clusters).

In certain extreme cases, for instance when an island is built around a bridge pier to protect it from ship impact, equipment also can be considered as an additional structure.

8.3.3 Even when danger is admitted, necessary endeavors to restrict the consequences of the accident (certain explosions or impacts) lead to select strong structural forms, apt to allow for various redistributions of load-effects after the accident. This selection is linked to the selection of a particular partial model of destruction, such as mentioned in section 7.

This model, in turn, is linked to the selected form. Thus, in a gunpowder factory, preference is given to a lightweight roof topping heavy walls, so that, in the event of an explosion, the damages are concentrated in the roof.

Robustness as a safety procedure is the object of a detailed analysis in the Structural safety Manual (C.E.B. Bulletin 128, paragraph 11.8).

8.4 Loads due to supports and foundations.

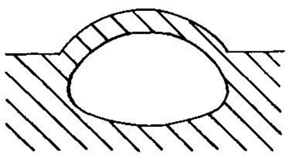
They are as a rule effects of other actions, not independent actions. But, whatever these other actions, human activity interferes specifically with them, through geometrical options, which often have very important influences on the structural form, since the function of structures consists primarily in transferring to supports and foundations the loads applied to buildings.

When such choices are made, the forces applied by the supports to the structures and reciprocally become fixed or almost fixed. The forms of the supports and of the supported structure may then be selected in such a way that fairly direct transfers of loads up to the foundations are obtained.

## 9. PARTICULAR PROBLEMS OF RELATIONSHIPS BETWEEN m.m.l. AND SELECTION OF THE STRUCTURAL FORM.

### 9.1 Case of certain permanent loads :

It was stated in paragraph 2.3 that the principal object taken into consideration would consist in the influence upon the selection of structural form of such m.m.l. as constitute data of the form selection; in the other cases, form selection and the corresponding m.m.l. interact, as is the case, in particular, of structure self weight. Such is the case also for a permanent load applied to the structures, during erection stages notably. Thus for tunnels under mountains, their shape and building procedures reduce the earth pressure applied to the inner covering with respect to its possible magnitude, to a considerable extent.



It is also necessary, sometimes, to create entirely artificial permanent loads, in order to carry out a selected form; the top of a flexible metal duct, for instance, must be loaded on backfilling, to prevent grave deformations of the duct, leading to collapse.

Inversely, in the case of settlement of mining ground, it may be advisable to contrive a flexible form, apt to resist imposed deformations.

### 9.2 Loads coming from maintenance works.

As well as building works, the works of maintenance of buildings give rise to application of man-made loads. Both lead to situations different of the durable situations; from our present point of view, this means that, at different times, the structures must withstand loads different in nature and in layout. The fact that several such situations must be taken into account leads to conclusions consistent with those obtained in taking free actions into account, with respect to the selection of structural forms.

## 10. CONCLUSION : LOGICALITY OF THE SELECTION OF THE FORM AND m.m.l.

The logicality of the selection of the form is quite complex : the data, constraints, and reasons to such a selection are very numerous and diverse; they interact upon one another and on the choice itself (sole, or almost sole decision) to lead to the so-called design (more precisely in French : la conception)

In this logic, the place of m.m.l., though far from negligible, is often rather modest; at any rate, they are not a major element of the diversity of forms. In this selection, m.m.l. do not intervene directly as a rule, but through their models and the designer, who according to the cases enjoys very unequal degrees of freedom with respect to the actions themselves.

## 11. ACKNOWLEDGEMENTS.

The authors wishes to thank Mssrs. G. DARPAS, N. ESQUILLAN and R. PERZO, whose ideas contributed to the elaboration of this report.