EUROPEAN PRESTANDARD
PRÉNORME EUROPÉENNE
EROPÄISCHE VORNORM

ENV 1991-2.6

Eurocode 1 : Basis of design and actions on structures

Part 2.6 : Actions during execution
Background document

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Section 1 General

1.1 Scope

1.1.1 Scope of ENV 1991 - Eurocode 1

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1.4 Definitions and terminology

1.4.1 Definitions

Auxiliary construction works (ouvrage auxiliaire, Hilfskonstruktion) : Any construction works associated with the construction process and removed after use (e.g. scaffolding, propping system, cofferdam, bracing, launching nose, etc.).

1.4.2 Terminology

Construction loads (charges de construction, Baulasten) : see definition given in 4.8

Launching nose (avant-bec, ...)

Precamber (contre-flèche, ...)

Travelling form (équipage mobile, ...)

1.5 Notation and symbols

Section 2 Classification of actions

In prestressed structures, attention is drawn to the fact that tendons, which are assumed to act in bond in persistent design situations, may be already bonded or unbonded in the relevant design situations during execution.

The elasticity of the propping system may have unfavourable effects, especially in the case where prestressing is designed to compensate the whole self-weight of concrete elements.

A similar position may be adopted, in a general manner, for everything which is put down by a crane.

Section 3 Design situations and limit states

3.1 General - identification of design situations

Change of structural systems may occur e.g. when simple span girders are built in, which are coupled at a later stage to be continuous. It may be relevant to take into account the residual stresses from heavy machinery when its weight is applied in one static scheme and removed in another one.

Cofferdams and scaffoldings are examples of auxiliary structures for which transient and accidental design situations should be defined where such situations may have an influence on the design requirements for the construction works under consideration.

3.2 Serviceability limit states

3.3 Ultimate limit states

3.4 Assessment of data for design situations

The major problem concerning the choice of characteristic values of variable actions, especially climatic actions, for transient design situations is related with the possibility of defining these characteristic values on the basis of mean return periods shorter than those adopted for persistent design situations. Hereafter are proposed some developments from Ir. Gl. H. Mathieu, convenor of the Horizontal Group for Bridges.

1.- Position of the problem

Is it acceptable or not, and how much, to reduce the characteristic values of variable actions during transient design situations ?

This question is often posed, for practical reasons, because the common sense considers unlikely that rather high values of these actions be reached during short periods such as, for example, design situations during execution, and taking these values into account may in some cases be very expensive.

The problem is not, at the present stage, mentioned in " Basis of Design " (section 4) but it is evoked in a document established by the JCSS[1] and published as CEB Bulletin 191 where one can read (6.2.1) that for variable actions the characteristic value is " chosen with regard to the design situation under consideration ".

It is not mentioned in ENV1991 Parts 2.3 (snow loads) and 2.4 (wind actions) but this last one provides formulae to be used for the assessment of wind velocities for various mean return periods. Part 2.6 of ENV 1991 specifies, on the basis of some collected opinions and of what can be read in other Parts of this Eurocode :

- for wind forces a mean return period of 10 years (instead of 50 years) for situations not longer than 1 year ;

[1] Joint Committee for Structural Safety

- for snow loads a value to be chosen not smaller than 50% of the value for persistent situations.

Finally, ENV 1991-3 (Traffic loads on bridges) specifies numerically a reduction (4.5.3), but only for traffic loads on road bridges - and therefore for transient design situations other than those to be considered during execution.

At national levels, rules can sometimes be found. For example, the French code for road bridges (dated 1971) specifies that in common cases wind pressure may be reduced during execution to 50% for phases of less than three months, 62,5% for longer phases.

2.- General considerations for the approach of the problem

The following notation and definitions are used in the following developments:

$Q_{k,pers}$	characteristic value of a variable action for persistent design situations
$Q_{k,trans}$	characteristic value of a variable action for transient design situations
T_{dwl}	design working (or service) life of the structure
T_{trans}	duration of a transient design situation
$T_{Q,pers}$	mean return period of the characteristic value of a variable action for persistent design situations
$T_{Q,trans}$	mean return period of the characteristic value of a variable action for transient design situations.
$T_{Q,real}$	real (or physical) mean return period of the characteristic value of a variable action

To assess the right characteristic values for transient design situations by referring to characteristic values for persistent design situations, it should be taken into account :

- the foreseeable duration of the various transient design situations,
- the additional information that may be collected concerning the magnitude of the actions, depending on the duration and dates of the transient design situations,
- and the identified risks, including possibilities of intervention.

Although the design working lives do not intervene directly in the choice of $Q_{k,pers}$, the comparison of the characteristic values should obviously be based on a comparison of the respective duration T_{trans} and T_{dwl} : For any high value Q* of Q the probability of exceedance is indeed practically proportional to this ratio as far as the action process can be considered as stationary :

$$\frac{\Pr\text{ob}(Q > Q^*) \text{ during } T_{trans}}{\Pr\text{ob}(Q > Q^*) \text{ during } T_{dwl}} \cong \frac{T_{trans}}{T_{dwl}}$$

For climatic actions the additional information is generally linked to :

- the seasonal aspect, for periods that can be measured on a month-scale ; when it can be taken into account, 3 months may generally be considered as the nominal value of T_{trans} ;

- and/or the possibility to get reliable meteorological previsions, for periods that can be measured in a few days or in hours ; when appropriate, 1 day may generally be considered as the nominal value of T_{trans}.

For man-made actions, the additional information may generally be linked to the control of the actions and of their effects ; the duration is then not a major parameter for the comparison.

All such information is generally taken into account in the practice, but it can hardly be codified in a general form ; it is also commonly more or less subjective.

In general, 1 year may be accepted as the nominal value of T_{trans} ; at this scale of time, the action process may be considered as stationary and the same as for persistent situations.

On the way to take the risk into account, the basic principles are generally applicable. However, for their application, data are in most cases widely specific ; in particular it is often possible to prevent or to reduce the consequences of an initially unexpected event, which may justify to accept a priori higher probabilities for such unfavourable events.

Some other differences between transient and persistent design situations may still be noted :

- for a variable action whose maxima follow a Gumbel's law, the coefficient of variation is higher for a shorter period than for T_{dwl} (the standard deviation is independent of the period, but the mean value is lower) ; as a consequence γ_F values should be somewhat increased ;
- on the resistance side, during execution the concrete strength has not yet reached its final value (unfavourable effect), but the degradation of materials, especially of steels, has not yet occurred (favourable effect).

3.- Numerical assessment of characteristic values for one-year transient design situations

a) This assessment will be done in terms of mean return periods, which is valid for stationary processes (beyond design working lives, mean return periods remain valuable mathematical measurement tools).

For the design working lives T_{dwl} (see 2.4 in ENV 1991-1), which are comparable to the duration of the persistent design situations, ENV 1991-1 in 4.2(8) does not specify precise rules for the mean return period of characteristic values. It states only, as an assumption, in most cases, an annual probability equal to 0,02 for the exceedance of the associated " time varying part " (we might say, the physical agent, e.g. the snow height on the ground or the wind speed), which means a mean return period $T_{Q,pers}$ = 50 years[2].

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[2] It cannot be ignored, indeed, that for climatic actions, action models always include important hidden additional reliability margins, so that the real mean return periods $T_{Q,real}$ corresponding to codified $Q_{k,pers}$ values generally amount to several centuries (this is obviously also the case for imposed actions, for buildings as well as for bridges). However, since the model uncertainties of any given action are the same in the assessment of $Q_{k,trans}$ and of $Q_{k,pers}$, it can be admitted that for transient situations it is sufficient to assess a nominal mean return period $T_{Q,trans}$ to be substituted to $T_{Q,nom}$.

b) Let us now compare acceptable failure probabilities for transient and persistent design situations.

It is difficult to perform scientifically based calculations of failure probabilities as functions of time : failure probabilities during the individual years of a persistent design situation are indeed not mutually independent (many data are the same : permanent actions and material properties ; besides, in the usual case of an existing structure some distributions of basic variables can be progressively truncated) ; besides failure probabilities during transient situations are not fully independent of failure probabilities during persistent design situations in spite of the involvement of some specific basic variables. However in CEB Bulletin 202 (5.5.2 p. 81 to 85), Lars Östlund demonstrates that in common cases, the mutual dependency has very significant consequences on the reliability level only when permanent actions G are dominant by comparison with variable actions Q (dominant in influence, not in variability)[3].

If we assume roughly a full independence of failure probabilities during transient and persistent design situations, it appears that, by reducing for transient situations the mean return periods proportionally to the duration of the situations (i.e. multiplying them by T_{trans}/T_{dwl}) one gets the same failure probability during one transient design situation and one persistent design situation.

However, if equal failure probabilities were accepted for transient and persistent design situations, it immediately appears that, in spite of the mutual dependency of annual failure probabilities, taking into account a persistent situation as consisting of e.g. 50 transient situations would considerably increase the cumulative failure probability.

Conversely, if Q_{ktrans} were taken equal to Q_{kpers}, the number of failures during transient situations would obviously be very low by comparison with what is accepted for persistent situations.

Therefore, it seems that *an intermediate choice would be reasonable.*

c) To a certain extent, the choice of characteristic values for 1 year nominal transient design situations may be considered to be analogous to the choice of combination values for persistent situations. The format of the combinations is justified by Turkstra's principle and by calculations presented in CEB Bulletin 127 (chapter 7) : for the choice of a combination value $\psi_0 Q_{2k}$ the objective is that the joint characteristic effect of $(Q_1 "+" Q_2)$ has the same probability of exceedance as the characteristic effect of Q_1 alone in the absence of Q_2, which in the practice means that the effects of $Q_{1k} + \psi_0 Q_{2k}$ and of Q_{1k} alone should correspond approximately to the same mean return period. We have indeed, for two actions, two combinations, and therefore for the joint effect a mean return period divided by two, but in the practice ψ_0 factors are chosen so that all possible influence ratios of Q_1 and Q_2 are enveloped (see CEB Bulletin 127) and in any case the difference on failure probabilities is not significant for the reliability format.

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[3] By common cases one has to understand that the variability of G, Q and of the resistances have magnitudes and influences that are commonly met (ranges of common values are given in the Bulletin).

In the calculations of ψ_0 values, the simultaneity of Q_1 and Q_2 is taken into account and their variability is combined by considering their distributions in time (Ferry Borges' model generally). The result is that $\psi_0 Q_{1k}$, corresponding to a conditional probability of exceedance during a persistent design situation, may also be interpreted as corresponding to an accepted probability of exceedance of Q_2 during the bigger elementary interval of Q_1 and Q_2, i.e. during a period of time smaller than the design working life.

An interesting example can be found in characteristic combinations of actions for buildings, where the biggest elementary interval is generally the elementary interval of the imposed load Q_1, the order of magnitude of which is commonly 1 year (sometimes 5 years which corresponds to furniture loads), and $\psi_0 Q_{2k}$ are combination values of climatic actions (snow or wind). These combinations values are acceptable values of these climatic actions during 1 year, which is equal to the duration of transient situations we are considering now. If we accept this analogy, we can choose these values as characteristic values during these transient situations, independently of the type of construction works.

It can be noted that the choice of ψ_0 factors may also be influenced by some liability considerations : it has been considered in France that judicial courts might consider that any value of an action smaller than its codified characteristic value should be considered as normally foreseeable, the codified values constituting, from a general point of view, a boundary between reprehensible and non-reprehensible liabilities. As a consequence the product $\gamma_F \psi_0$ should not be taken smaller to 1 in ULS verifications. The same rule might be assumed for the characteristic values during transient situations.

d) Numerically, for climatic actions, if as given in " Basis of Design " for buildings, the boxed value $\psi_0 = 0{,}6$ is accepted, it can be easily calculated that :

- for an action the maximum of which in 50 years has a coefficient of variation equal to 0,2 (which is commonly accepted for wind and snow), and is distributed in accordance with a Gumbel's law, the nominal mean return period of $\psi_0 Q_{1k}$ is approximately equal to 5 years, i.e. 0,1 $T_{Q,nom}$ (on the basis of the combination values proposed in CEB Bulletin 127, the real mean return period would be about 0,15 $T_{Q,real}$) ;
- the product $\psi_0 \gamma_Q$ is 0,9, not much smaller than 1.

This reduced mean return period is not very different (slightly smaller) from the pragmatic values mentioned in § 1. Therefore, it seems that the order of magnitude of these results may be accepted for 1 year transient situations, without excluding further possible reliability differentiation.

5.- Conclusions

Characteristic values of variable actions for transient design situations may be chosen smaller than for persistent design situations. The values to be adopted cannot be accurately demonstrated : they have to be chosen on the basis of engineering judgement helped by analogies and comparative calculations.

For 1 year transient design situations, mainly for climatic actions, five years nominal mean return periods (instead of 50 years) seem to be acceptable. For shorter transient situations (e.g. 3 months or 3 days) characteristic values may be more reduced on the basis of additional information from various origins.

In some cases any reduced characteristic value may have to be reconsidered for optimisation of the reliability level.

Section 4 Representation of actions

4.1 Self-weight of structural and non-structural elements, and permanent actions caused by the ground

4.1.1 General

Non-uniformly distributed self-weight may occur when cast in situ of concrete is provided spanwise or sectionwise. The intended organization of concreting and its control are essential.

For the assessment of vertical forces only, the characteristic density[4] of fresh concrete is assessed by increasing the design characteristic value of the density of concrete by 1,00 kN/m^3, in accordance with EC1 Part 2.1. After drying, this enhanced density is decreased to its normal design characteristic value.

4.1.2 Buildings

Attention should be paid to concrete floor with early-aged material properties (smaller strength, cracking, deflection).

If the load imposed on a partially cured slab is higher than its capacity, the construction procedure must be changed to either reduce the slab loads or increase the concrete strength at the age at which the maximum load occurs.

4.1.3 Bridges

No information is available in current European standards concerning the assessment of self-weight of bridge parts during execution. This problem is very important for prestressed concrete bridges built by the cantilever method or for the launching of steel bridges.

What is proposed in the draft EC1-2.6 is in accordance with Eurocode 2 part 2 : the design values of the favourable and unfavourable parts of self-weight are respectively :

$$G_{\mathrm{d,inf}} = 0{,}95 G_{\mathrm{m}} \qquad G_{\mathrm{d,sup}} = 1{,}05 G_{\mathrm{m}}$$

where G_{m} is the mean value of self-weight. Provisions to achieve these reduced values are specified in the relevant design Eurocodes. These design values are relevant not only in static equilibrium verifications, but also in other kinds of verifications, e.g. corresponding to uplift situations. Fig. 4.1 gives examples of situations including static equilibrium aspects.

[4] The word “ density ” is used as in ENV 1991-2.1 “ Densities, self-weight and imposed loads ”. It is strictly mass per unit volume.

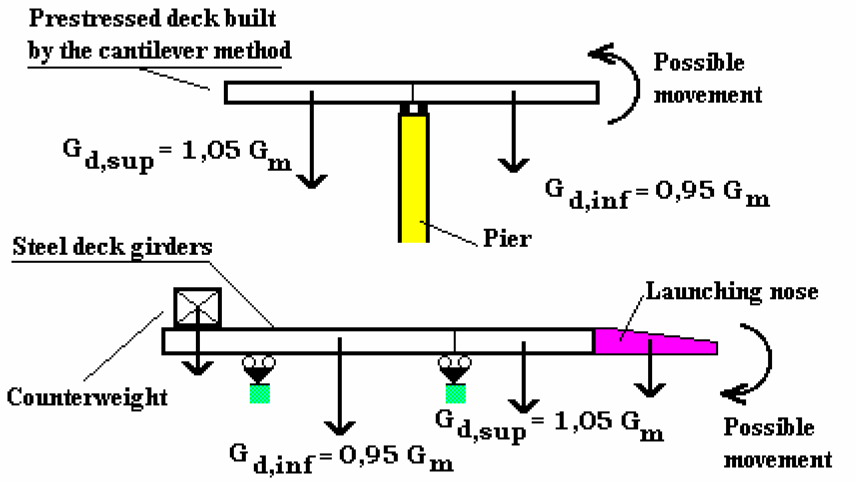

Figure 4.1 - Examples of situations including static equilibrium aspects for bridges

But less severe conditions may be specified if, for example, support reactions are accurately controlled.

4.2 Prestressing, intentional imposed deformations and settlements

4.2.1 Prestressing

In the case of prestressed concrete floors with unbonded tendons, special attention should be paid to the early-aged material properties and to the loadbalancing forces in combination with minimum variable floor loads during execution. In general, the limit state to be checked is the prevention of unacceptable cracks or crack widths.

4.2.2 Predeformations

4.2.3 Soil settlements

4.2.4 Unevenness of temporary bearings

This problem is mainly met during construction of bridges built by the incremental launchin method. But the characteristic values of deflections may be modified if particular control measures are taken during execution. Attention is drawn to the fact that box-girder bridge decks are very sensitive to a transverse level difference of bearings on abutments.

4.3 Temperature and shrinkage ~~effects~~ actions

Note : Examples are :
- drying in the open air ;
- shrinkage due to a difference in time between casting one concrete element to another element that has already hardened.

The limit state to be checked is the prevention of unacceptable cracks or crack widths, especially in the case of composite steel and concrete structures.

Note : During execution, and depending on the bridge type, bearings may be temporary or permanent : these two kinds of bearings have generally different behaviours.

4.4 Wind actions (Q_W)

Sufficient information has to should be collected or provisions have to should be undertaken to ensure that the characteristic wind force is not exceeded during the critical operations. Such information can be obtained from weather forecasts of the nearest meteorological station and local wind measurements.

4.5 Snow loads (Q_{Sn})

4.6 Water actions (Q_{wa})

Static pressures may result in buoyancy effects and lateral forces.

Note : An example of exception is tide effects.

In case the values of water actions assessed in accordance with the project specification may can be exceeded by floods or other accidental events during the construction period, these effects should either be considered in the design, or provisions should be planned to avoid the effects.

Such provisions may be flooding of excavations, etc.

Instability failure due to water ponding on roof structures may be avoided by designing for a minimum stiffness and by accurate construction.

4.7 Ice loads

Ice loads should be considered in the design where either by ice on water or icing of cables or other structural parts of masts and towers such actions have to be considered in the execution stages.

4.8 Construction loads (Q_c)

4.8.1 General

Where construction loads are free actions, they should be placed arranged in the most adverse position for each individual verification.

4.8.2 Specific construction loads for buildings

In the French practice, specific loads are to be considered only in the following cases :

- concrete buildings constructed with several erection steps :
    * floors with ribs and blocks,
    * floors with thin floor plates,
    * hollow core floors,
    * composite floors

- steel roofs on which waterproofing works have to be taken into account.

1.- Ribbed and composite slab floors

The design situation to be considered corresponds to a local accumulation of fresh concrete on any rib during concreting of the floor, including a dynamic magnification. The associated limit states are :

- a limit state, similar to an ultimate limit state of resistance, corresponding to the failure of the loaded rib ;
- a serviceability limit state corresponding to a limitation of the concrete tensile stress.

On floors made of ribs and blocks (Fig. ...) or on composite floors (Fig. ...), a concentrated load is taken into account on any rib, located at the middle of each span between two temporary supports (Fig...).

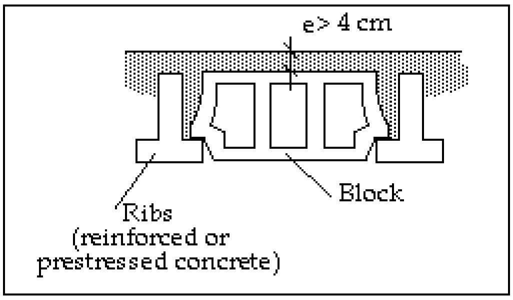

Fig. ... - Typical floor with ribs and blocks

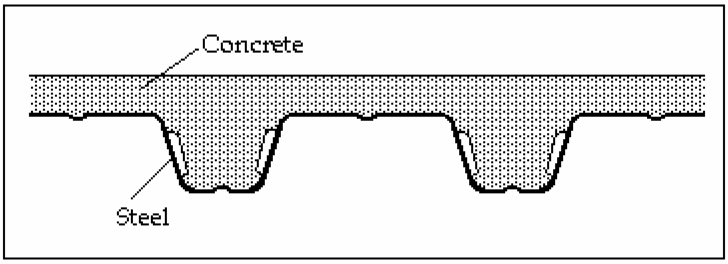

Fig. ... - Example of composite floor

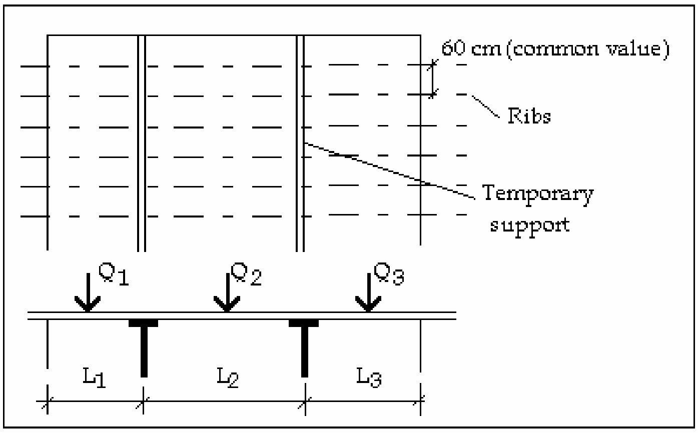

Fig. .....

This loading system should be cumulated with the weight of fresh concrete corresponding to the design thickness of the floor. Its intensity is given by :

$$Q_i = 0{,}5 \times L_i \quad (kN)$$
$$Q_i \geq 1\ kN \quad \text{in all cases}$$

L_i is the span length between to temporary supports as defined in fig. ....

The fundamental combination of actions is : $1{,}5G + 2Q_k$
where Q_k designate the characteristic (nominal) value of the concentrated loads (Q_i).

The maximum bending moment resulting from this combination is compared to the ultimate bending moment with a γ_m factor equal to 1, the compressive strength of contrete being taken at 7 days.

For this verification, all spans between two supporting lines are considered as simply supported spans for ribbed floors.

The characteristic combination of action is : $G + Q_k$
For this limit state, the tensile stress in concrete is limited to 4 Mpa for ribbed floors.

2.- Floors with thin floor plates

The design situation to be considered corresponds to a local accumulation of fresh concrete during concreting of the floor, including a dynamic magnification. The associated limit states are :
- a limit state, similar to an ultimate limit state of resistance, corresponding to the failure of any floor plate ;
- a serviceability limit state corresponding to the limitation of the tensile stress.

On floors built with thin floor plates (Fig. ...), a knife-edge load is taken into account, located at the middle of each span between two temporary bearing arrangements.

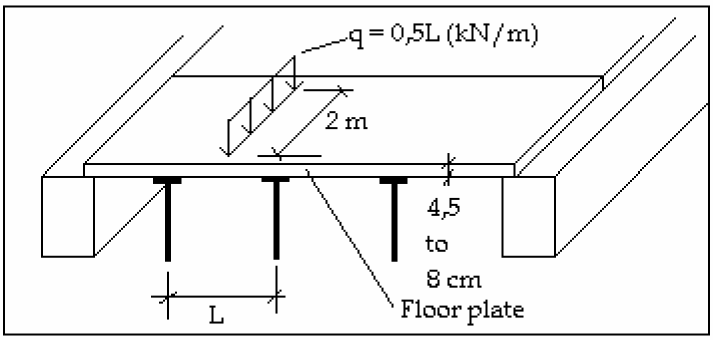

Fig. ...

The nominal value of this load is equal to 0,5L (kN/m) with a miminum value equal to 1,0 kN. L is the span length between two bearing lines.

The fundamentral combination of actions is : $1{,}5G + 2Q_k$
where Q_k designate the characteristic (nominal) value of the knife-edge load.

The bending moment resulting from this combination is compared to the ultimate bending moment with a γ_m factor equal to 1, the compressive strength of contrete beeing taken at 7 days.

The ultimate bending moment is calculated with a plate thickness e* equal to :

- e* = e - 0,8 or 0,5 cm when the plate is supported by a temporary bearing system,
- e* = e - 1,2 or 1 cm when the plate is not supported by a temporary bearing system

e is the nominal thickness of the plate.

Le characteristic combination of action is : $G + Q_k$

For this limit state, the tensile stress in concrete of the floor plate is limited to :

- 0,75 f_{ctk} where execution is self-controlled,
- 0,5 f_{ctk} where execution is not self-controlled.

3.- Hollow core floors

The design situation to be considered corresponds to a local accumulation of fresh concrete on the slab. The associated limit states are :

- a limit state, similar to an ultimate limit state of resistance, corresponding to the failure of any hollow core element ;
- a serviceability limit state corresponding to the limitation of the tensile stress.

On hollow core floors (Fig. ..) uniformly distributed load is taken into account on the whole surface of an element.

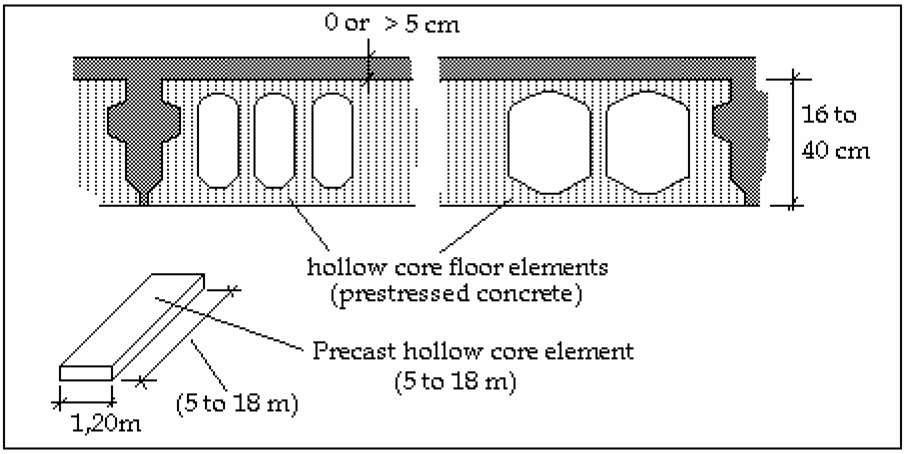

Fig. ...

This load should be cumulated with the weight of fresh concrete corresponding to the nominal thickness of the floor. Its intensity is equal to 1 kN/m^2.

The combinations of actions to be considered are the same as combinations for persistent situations, the variable action being the action previously defined.

4.- Steel decks

Steel decks are loaded with either :

- a uniformly distributed load equal to :
  - * 1 kN/m^2 with topping
  - * 0,5 kN/m^2 without topping ;
- a knife-edge load equal to 2 kN/m located at mid-span between two steel ribs.

4.8.3 General overview

From a general point of view, it could be envisaged not to define any load or verification rule for the construction phase of buildings, and to consider that it is the responsibility of the designer to define appropriate construction methods to ensure the required safety and reliability levels for the final stage.

Such a point of view cannot be acceptable because the consequences of a general or local failure of a building under construction entails risks to human lives.

Moreover, if we postulate that the failure of floors defines the failure of the building, and thus that the level of safety associated with the various kinds of floors defines that of the building, it seems reasonable to adopt the same safety levels for buildings during construction as those associated with service. Therefore, there is no obvious reason why the risk should be any greater during construction than it is during service.

These considerations entail that, at least, a frame should be defined, giving verification rules for the execution stage, calibrated in order to ensure acceptable safety and reliability levels.

In the construction of multistorey concrete structures, a newly poured floor is generally supported by a system of shores and reshores resting on lower floors that have not necessarily atteined their full strength. In the case of concrete buildings during execution, the assurance of the safety requires careful consideration of two main problems :

- the adaquacy of the formwork and shoring systems to transfer the loads during various stages of construction,
- the ability of the partially cured slabs to resist the applied loads.

Analogous considerations may be developed for other kinds of buildings.

Numerical data[5], resulting from measures and investigations on construction sites. During the visits, data on equipment, material storage and worker loads were collected for two stages of construction : (1) before the concrete slab has been poured and (2) after the slab has been poured and preparations for the construction of the next floor.

To analyse the collected data, the surveyed floors were divided into successive sets of grids of various sizes. The following table shows the observed mean strutucral loads,

[5] " Cast-in-Place Concrete in Tall Building Design and Construction " - Council on Tall Buildings and Urban Habitat Committee 21 D. Mc Graw-Hill, Inc. Chapter 2 : construction loads.

transformed into an equivalent uniformly distributed load, and the load intensities at 90, 99 and 99,5% probability levels for different grid sizes.

Grid size	Mean load	90% load	99% load	99,5% load
m^2	kN/m^2	kN/m^2	kN/m^2	kN/m^2
2,32	0,31	1,08	2,93	3,34
5,95	0,30	0,92	2,0	2,39
9,25	0,29	0,80	2,18	2,68
20,9	0,30	0,73	1,58	1,94
37,16	0,28	0,72	1,43	1,46

It can be noticed from this table that the mean load is rather independent of the loaded area and its magnitude is rather low. But the coefficient of variation is very high, ranging from about 1,00 to 2,00, which explains the high values corresponding to fractiles of 10, 1 and 0,5%.

Assuming that the distribution law of these loads is close to a Gumbel type, it is rather easy to determine a characteristic value corresponding to a 5% fractile : for example, for a grid size of 9,25 m^2, this value is 1,23 kN/m^2.

Other studies[6] have suggested higher mean construction variable actions, but with a lower value of the coefficient of variation. Nevertheless, it turns out that :

- the order of magnitude of the characteristic vertical uniformly distributed load on floors during execution of buildings is close to 1 kN/m^2 ;
- the geometrical variability of loads during execution should be covered by an overloading on a limited area.

From a pragmatic point of view, the objectives of verifications during execution are :

1. To avoid accidents , perhaps leading to progressive collapse of the building, with losses of human lives due to structural failure or loss of static equilibrium ;
2. To avoid excessive long-term deflection after the completion of the construction, which might not be fully consistent with the serviceability criteria defined for the design.

4.8.3 Specific construction loads for bridges

Specific loads for bridges have to be taken into account in specific cases like for bridges built by the cantilever method or by the incremental launching method.

1.- Bridges built by the cantilever method

Generally, no special rules are defined for prestressed bridges built by the cantilevered method during their erection. The assessment of stresses and deformations is carried out on the basis of serviceability limit state combinations including :

- permanent actions (weight of the structure),

[6] “ Partial Factor Design for Reinforced Concrete Buildings during Construction ” by Ashraf M. El-Shahhat, David V. Rosowsky and Wai-Fah Chen - ACI Structural Journal - July-August 1994.

- variable actions (essentially the weight of construction systems like movable scaffoldings, launching girders, etc. , which are known at the design stage).

The main situation requiring specific rules is the possible loss of stability of an arm.

Accidental design situation (ULS)

The accidental design situations to be considered are :
- the fall of a movable scaffolding during its movement,
- the fall of a precast segment before application of the final prestress.

If safety may be considered as ensured during such situations, even with a low safety margin, the static equilibrium for a non accidental situation is assumed to be covered. The associated limit states to be considered are :
- the non decompression of the less loaded line of bearings, when the flails are built over two bearing lines, or
- the resistance of stability systems of the structure (e.g. temporary prestressing tendons - see fig...).

Design loads during execution

In general, the design loads during execution include :
- The self-weight of the structure G, taken into account with two values G_{max} and G_{min}, as defined below ;

- Variable actions Q_c, including :
  - loads corresponding to construction systems (cranes, movable scaffoldings, etc.) which are known and noted Q_{c1},
  - supplementary loads, which are not specifically identified, represented by a uniformly distributed load q_{c2} and a concentrated load Q_{c2},
  - a difference of vertical pressure due to wind between the two parts of a flail, represented by a uniformly distributed load q_w ,
  - and possibly a non uniforme snow load.
- An accidental action F_A, corresponding to the fall of a movable scaffolding or of a precast segment before application of the final prestress, including dynamic effects.

The load arrangement of the variable actions is taken in order to get the most unfavourable effects.

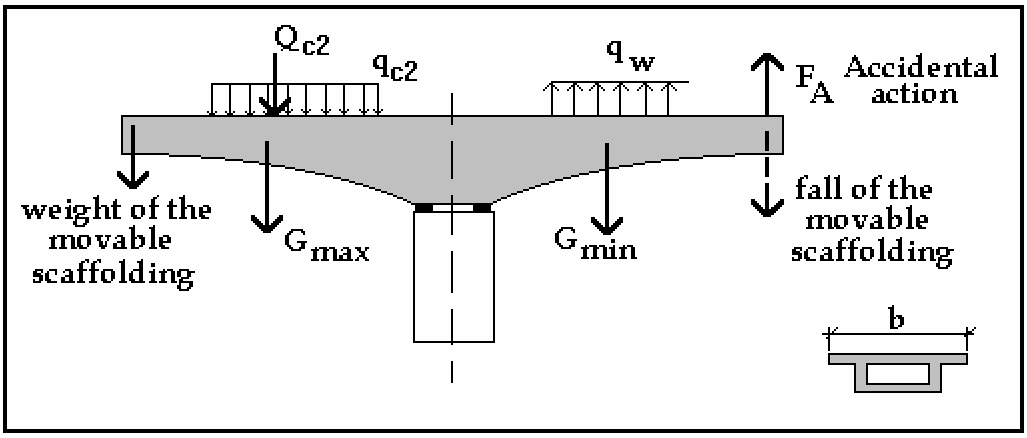

Fig....

- $G_{max} = 1{,}02G_m$ $G_{min} = 0{,}98G_m$
- $q_{c2} = 200N/m^2$ $Q_{c2} = (50 + 5b)kN$ (b = deck width)
- $F_A = \begin{cases} \text{weight of the movable scaffolding, or} \\ \text{weight of a segment} \end{cases}$
- $q_w = 100 \text{ to } 200N/m^2$

In the French practice, the combinations of actions are built as follows :

$$0{,}9(G_{max} + G_{min}) + 1{,}25Q_k$$
$$1{,}1(G_{max} + G_{min}) + 1{,}25Q_k$$
$$0{,}9(G_{max} + G_{min}) + F_A + Q_k$$
$$1{,}1(G_{max} + G_{min}) + F_A + Q_k$$

where Q_k represents the variable actions Q_{c1}, Q_{c2}, q_{c2} and q_w.

Assessment of stability systems

* For provisional bearings, reference is made to BAEL rules.

* For stability tendons, the ultimate tensile strength is taken equal to $\frac{f_{pk}}{1{,}15}$.

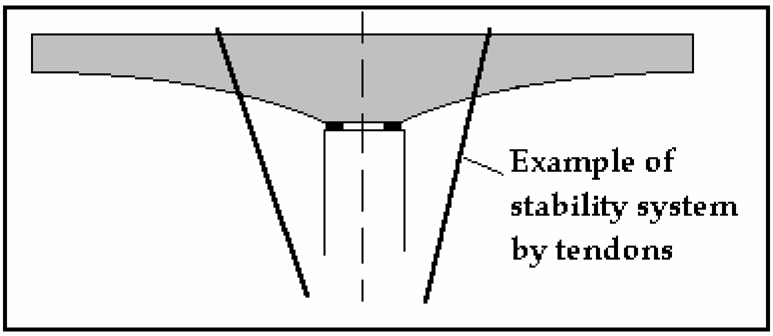

Fig... - Stability system by tendons

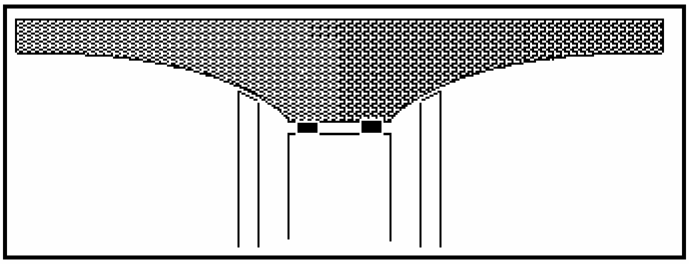

Fig.... - Example of stability system with (steel or concrete) columns

Figure ... gives an example of load arrangement corresponding to the loading system under consideration.

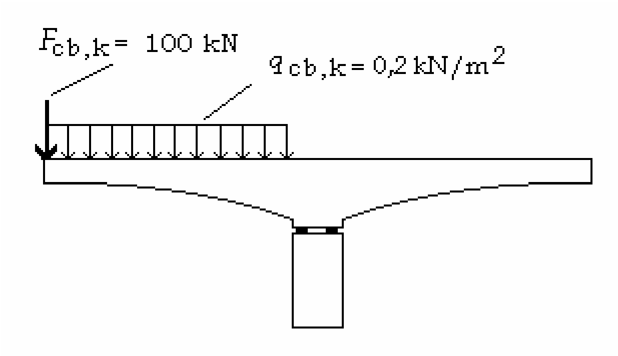

Fig.... - Example of load arrangement during execution

If some local verifications are needed, the impact area of $F_{cb,k}$ should be defined.

The values will depend on the method of execution. For example, the friction ratio depends on the anticipated launching method : rolling, sliding of teflon on stainless steel, sliding on lubricated steel, etc.

2.- Prestressed bridges built by the incremenbtal launching method

Prestressed bridges can be launched with several methods :

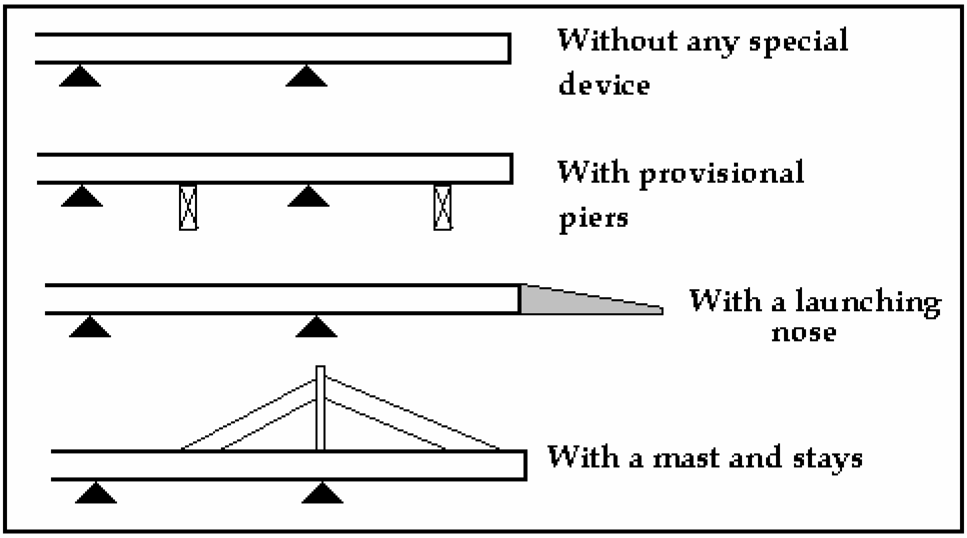

Fig.... 4

Prestressed concrete bridges built with the incremental launching method are designed in such a way that consideration of loss of static equilibrium is generally irrelevant. The design situations to be considered are mainly related to serviceability limit states, with temporary prestressing tendons.

The following limit states are considered :

- no tensile stress in any cross-section under a quasi-permanent type combination (defined below), the prestressing force being taken into account with its medium value, or
- limitation of tensile stresses under a characteristic type combination (defined below), including variable actions. The prestressing force is also taken into account with its medium value.

The various loads and deformations to be taken into account during the launching stage are :

* The self-weight of the launched structure, including the weight of possible equipment, of the launching nose or of the stay system, and taken with its nominal value G.

* Thermal effects Q_t, corresponding to a temperature gradient of 6°C in the bridge deck.

Where temporary stays are used, specific rules concerning thermla effects are applied to these stays.

* Imposed vertical deformations of provisional bearing pads, defined as follows :

  a) ± 1 cm longitudinally for a single bearing line (all other pads are assumed to be at their theoretical level), or

  b) ± 0,25 cm transversally for a single bearing line (all other pads are assumed to be at their theoretical level).

Generally, cases a) and b) are chosen by the " client " for a particular project and the foundation settlements are ignored. These cases normally cover the vertical deformations due to a launching system with lifting of the deck (e.g. system Eberspächer).

The variable actions resulting from these imposed deformations are noted Q_s.

Sometimes, the following complementary rules are considered :

* The geometrical uncertainties concerning the precasting form may be takent into account through an angular difference between two precast segments equal to $0{,}5.10^{-3}$ rd.

* For the assessment of pier resistance, the horizontal force applied by the deck during the launching is taken equal to 10% of the concomitent vertical force.

At the SLS,

$$\sum_i G_i + P_m$$
$$\sum_i G_i + P_m + Q_t + Q_s$$

3.- Launched steel and composite steel-concrete bridges

The design situations to be considered during the erection stage are :
- a possible loss of general stability (static equilibrium),
- a possible loss of stability of girder webs and/or flanges (mechanical stability).

The associated limit states to be considered are :
- the loss of static equilibrium during erection,
- the ultimate limit state of mechanical stability of the structure.

In the French practice, the various loads to be considered for this limit state are :

* The weight of the structure G, taken with its nominal value, including loads of cranes, equipment, scaffolding, etc. and for which are distinguished :
  - the favourable part noted G_1,
  - the unfavourable part noted G_2.

* Where relevant, the weight of a counterweight, Q_c. An error of positioning of this counterweight, equal to 1 m, is taken into account for the definition of load arrangements.

* The weight of the launching nose, Q_n.

No variable action is generally envisaged. In particular, it is considered that no wind action with a significant value has to be considered because launching is a rather short operation.

In the current draft EC3 Part 2, an additional variable action is defined, representing loads due to persons and modelised by a uniformly distributed load q_s = 1kN/m^2, this load being applied on such areas so as to get the most adverse effects.

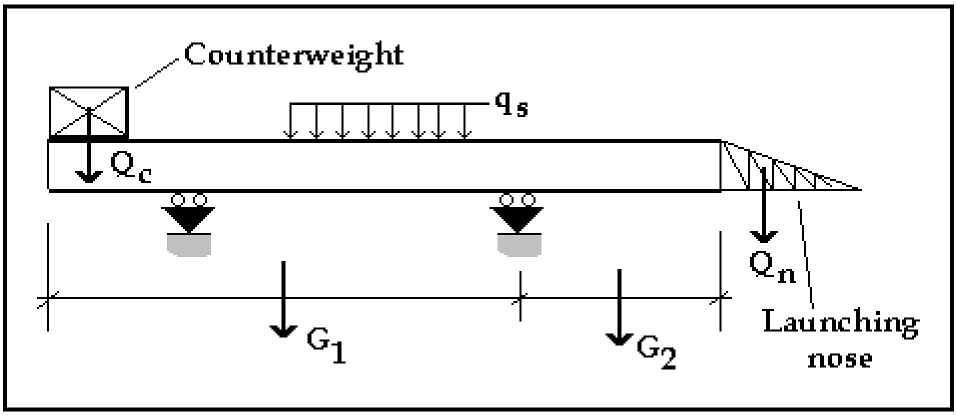

Figure 5

The same actions regarding the ultimate limit state of resistance should be considered as for the loss of static equilibrium.

The French practice consists in considering the following combination for the loss of static equilibrieum :

$$1{,}05G_2 + 0{,}95G_1 + 1{,}20Q_n + 0{,}80Q_c$$

(taking into account the error of positioning of the counterweight).

This equation should be supplemented with other variable actions (q_s, wind pressure, snow loads, etc...)

For the resistance ultimate limite states The verifications are performed in accordance with the French regulation for steel and composite bridges.

4.9 Seismic actions

A project specification for short term phases or local effects is generally irrelevant.

4.10 Accidental actions

4.10.1 Accidental actions for buildings

(2) Measures (inspection on site) should be taken to avoid :

- abnormal concentrations of building equipment and/or building materials on loadbearing structural parts ;
- water accumulation (e.g. on steel roofs).

Water accumulation on roofs is mainly possible in the case of a building which is quasi finished ; consequently, this situation is covered by the permanent situation.

4.10.2 Accidental actions for bridges

Note : This implies that the action effect of the fall of the travelling form is equivalent to a force equal and opposite to its self-weight.

Note[7] : In prestressed concrete bridges built by the cantilever method with prefabricated units, it should be envisaged the possibility that a unit falls before fixing by the final prestressing (Fig. 4.6).

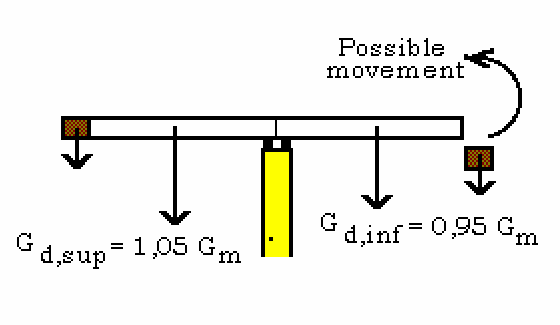

Figure 4.6 - Example of accidental situation (fall of a prefabricated unit)

For other vehicles, see ENV 1991-3. For any assessment of the impact force of a vehicle, its likely velocity should be taken into account.

[7] ~~Should this note be transformed into a flexible rule ?~~

Annex A (normative)

Basis of design - supplementary clauses to ENV 1991-1 for buildings

A.1 - Combinations of actions for various design situations

A.1.1 Simultaneity of variable actions

A.1.1.1 Mutual simultaneity of the various construction loads (Q_c)

A.1.1.2 Simultaneity of construction loads with other variable actions

A.1.2 Transient situation

A.1.2.1 Ultimate limit state of static equilibrium

A.1.2.2 Ultimate limit state of resistance

A.1.2.3 Serviceability limit states

A.1.3 Accidental situation

A.2 ψ factors for buildings

Annex B (normative)

Basis of design - supplementary clauses to ENV 1991-1 for bridges

B.1 - Combinations of actions for various design situations

B.1.1 Simultaneity of variable actions

B.1.1.1 Mutual simultaneity of the various construction loads (Q_c)

B.1.1.2 Simultaneity of construction loads with other variable actions

B.1.2 Transient situation

B.1.2.1 Ultimate limit state of static equilibrium

$$\sum G_{d,sup} \text{"+"} \sum G_{d,inf} \text{"+"} \gamma_P P_k \text{"+"} \gamma_{Q1} Q_{dst,1k} \text{"+"} \sum_{i>1} \psi_{0i} \gamma_{Qi} Q_{dst,ik} \qquad (B.1)$$

$$\sum G_{d,sup} \text{"+"} \sum G_{d,inf} \text{"+"} \gamma_P P \text{"+"} \gamma_{Q1} Q_{dst,1k} \text{"+"} \sum_{i>1} \psi_{0i} \gamma_{Qi} Q_{dst,ik} \qquad (B.1)$$

where :

- $Q_{dst,1k}$ represents the characteristic value of the dominant destabilizing variable action,
- $Q_{dst,ik}$ represents the characteristic value of the accompanying destabilizing variable actions.

P is a characteristic or a mean value depending, at the present time, on the project specification. Very varied situations may have to be considered during the execution of a bridge, for which formula (B.1) needs possibly amendments (e.g. for the verification of stabilization stays).

(2) For self-weight of structural and non-structural elements, $G_{d,sup}$ and $G_{d,inf}$ are assessed in accordance with 4.1.3.

(3) In general, where a counterweight is used, the variability of the action due to its self-weight should be considered. Unless otherwise specified, this variability should be taken into account :

- by applying a partial factor $\gamma_{G\,inf} = 0{,}8$ where the self-weight is not well defined (e.g. containers) ;
- by considering a variation of its project-defined location, with a value to be specified proportionately to the dimensions of the bridge, where the magnitude of the counterweight is well defined.

For steel bridges during launching, the variation of the counterweight location is often taken equal to ± 1 m.

(4) Unless otherwise specified :

- the γ factors for all variable loads should be taken equal to 1,35,
- $\gamma_P = 1$

B.1.2.2 Ultimate limit states of resistance

B.1.2.3 Serviceability limit states

B.1.3 Accidental situation

$$\Sigma G_{\mathrm{d,sup}} "+" \Sigma G_{\mathrm{d,inf}} "+" A_{\mathrm{d}} "+" P_{\mathrm{k}} "+" \psi_{01} Q_{\mathrm{c1,k}} "+" \sum_{i>1} \psi_{2\mathrm{i}} Q_{\mathrm{ci,k}} \quad \text{(B.2)}$$

$$\Sigma G_{\mathrm{d,sup}} "+" \Sigma G_{\mathrm{d,inf}} "+" A_{\mathrm{d}} "+" P \; "+" \psi_{01} Q_{\mathrm{c1,k}} "+" \sum_{\mathrm{i}>1} \psi_{2\mathrm{i}} Q_{\mathrm{ci,k}} \quad \text{(B.2)}$$

where :

$Q_{c1,k}$ represents one of the groups of construction loads defined in 4.8 (i.e. Q_{ca}, Q_{cb}, Q_{cc} or Q_{cd}).

For example, in the case of bridges built by the cantilevered method, some construction loads may be considered as simultaneous with the accidental action corresponding to the fall of a prefabricated unit.

B.2 ψ factors for bridges

Annex C (informative)

Construction loads for profiled steel sheeting