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Autor(en): **Ballio, G. / Castellani, A. / Finzi, L.**

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On the design of massive structures for nuclear plants

Über dem Proiekt der massiven Strukturen für nukleare Anlagen

Du projet de structures massives pour des installations nucleairs

G. BALLIO, A. CASTELLANI, L. FINZI
Istituto di Scienza e Tecnica delle Costruzioni
Politecnico di Milano (Italy)

1. INTRODUCTION

From the civil engineering point of view, the design of thermonuclear power plants raises particular problems, especially those concerned with stress analysis, structural detailing, and construction techniques.

In fact, the presence of highly radioactive substances in the vessel, the primary circuit, the fuel deposit and elsewhere gives overriding importance to the sealing of the building, even under the most severe loads. Furthermore, even if the distances between floors, or the centre-to-centre distances between columns and vertical walls in these buildings are similar to those in large modern factories, the loadbearing structures, slabs, beams, columns and walls are particularly thick. This is in part due to the requirements of biological shielding and the need to make the building missileproof, but it is also the consequence to a large extent of the vast loads involved. Inside the containment structures there may be pressures in the range of dozens of ton m^{-2} while the outside may be subject to meteoric or military activity that can be translated into pressures 10 to 30 times greater than those due to wind. Finally, there may be mass horizontal forces, that, further exasperated by the rigidity of the structure, can be four to six times greater than those laid down in Italy for 1^{st} class seismic zones. As a result, even for buildings of modest transversal and longitudinal dimensions, the effects provoked by changes in temperature and by shrinkage are significant.

On the whole, the structures of nuclear buildings are massive and heavily braced, or even they are of the box type. It follows that great importance must be attributed not only to the foundation soils but also to the soil-structure interaction.

There are also cases of buildings in which the size itself of the ground plan dimensions becomes a factor of importance, further increasing the effects of inelastic variations in volume. This is true for the turbo-pedestals which, for 1000MW, can be 80m long, and for foundation mats, which tend to be interconnected in order to control safely the effects of differential settlements, which in turn condition the behaviour of the ducts connecting the various blocks of the building.

Besides thermal stresses and concrete shrinkage, earthquakes, wind and hurricanes, and pressure accidents, there are also, of course, the effects of dead weight and working loads. Besides these, there are also the local effects, which may well be measured in hundreds of tons, such as jet forces, the effects of pipe whipping, the fall of equipments during erection or substitution (fuel cask drop) and the action of internal missiles.

When therefore a single member has to be checked, a suitable combination has to be worked out of the most unfavourable overall loading conditions together with the most unfavourable local loads.

There are no established criteria on which to base the formation of these combinations, since many of these events involved are so rare, or even new, that information is insufficient for a statistical approach.

As a result, when in doubt it is normal to check all the combinations of possible events, laying down, for any one structure, tests for dozens of loading combinations.

Thus the structural designer is obliged to design buildings on the basis of a selection of loading conditions that seem decisive, and then to check them for the envelope of all the very many combinations possible. This is essential, since the structures involved are always highly redundant in which, of course, the elastic weights of the parts play an important role in the state of stress.

The obvious consequence is that, even where adequate computational instruments are available, the final stress analysis may lead to results that are locally very different from those allowed for at the design stage. This implies strengthening the steel reinforcing, with resulting problems of congestion.

Anyway, these structures are generally highly reinforced because it is better to reduce as much as possible the stiffness of individual members in order to minimise the conspicuous effects of temperature and shrinkage on the one hand, and, on the other, the so-called "edge" effects that arise to guarantee compatibility with adjacent elements.

To sum up, for the civil engineer, nuclear power stations differ from other constructions in the following ways:

1) the need to ensure perfect sealing in the most severe conditions;
2) the importance of inelastic variations in volume;
3) the type of massive, box structures involved;
4) the size of the loads weighing on the ground;
5) the need to check the structure for a considerable number of loading combinations;
6) the congestion of the reinforcing.

The designer will thus have certain unusual problems in working out the behaviour patterns of reinforced concrete structures, in calculating and constructing them.

As to the basic structure of reinforced concrete buildings, since the elements involved are mostly subjected to two - and three-dimensional states of stress, two problems arise: defining the partialisation surface and the ultimate strength of the concrete, the latter having wide variations for three-dimensional states of stress; defining its behaviour when subjected to thermal stresses.

As to calculation, it seems necessary to turn to finite element processes, since these structures cannot be thought of as frames. Furthermore, as the computation methods become more sophisticated, in order to establish congruence between structural elements of different kinds and behaviour, strong local stresses arise. A non linear analysis thus becomes necessary to ensure a perfect seal in spite of these forces. However, since the design is generally conditioned by these problems of sealing, it is not necessary to take the non-linear analysis to the point of collapse.

Because of the large number of loading combinations, it seems tempting to computerise the design as much as possible, leaving it to a computer code to sort out the severest loading combinations and the necessary controls. The present authors have their doubts about this, since it would seem to lead to a formal respect for the controls to the detriment of sound design.

Lastly, the considerations already mentioned, but even more, the need to respect the regulations in the controls, often lead to a congestion of reinforcing, even when thick or bundled bars are used. In this situation, detail drawings and tight control over the erection become important.

2. GAP BETWEEN STRESS ANALYSIS AND DESIGN

More and better computer methods are gradually becoming available for the approach by finite elements.

Until recent times it was considered a success to be able to transform a problem from discrete to continuous, but today the finite element method leads to a reversal of this process.

Thus, for example, a circular slab of constant thickness subject to generic concentrated and distributed loads and resting on Winkler type soil, may be studied by dividing it into finite elements,each seen as perfectly homogeneous, elastic and isotropic.

A computer will deal with this problem,though with some effort, and gives stress and strain states which, since they refer to finite elements, have to be re-converted to the continuous system.

So parameters like displacements, which are of an integral nature, can be worked out with good approximation without artificial discontinuities. On the other hand local factors, i.e. the stress strain state, are given as discontinuities and averages.This makes their interpretation uncertain and personal.

The fact that computers can be used giving apparently exact solution makes this process extremely attractive, especially when very many loading combinations are involved as is in fact the case for nuclear structures.

However, the present authors would like to point out that there is still a lot of work to be done in this field. In fact,of the four blocks shown in fig.1, no.3 has received a great deal of attention, but much less work seems to have been done on no.2 and no. 4.

What this all comes to is that at the end of the operation the structural designer is faced with tens of thousands of figures from which it is difficult to form an overall picture of the real behaviour of the structure and so of the reliability of the result.As a process it is vitiated by a series of rules of thumb, both before and after, that make it like the mythical "colossus" with feet of clay and,in this case, a tutty neck.

This state of affairs is, of course, known to those doing research in this field, and attention is now being directed to improving the ways

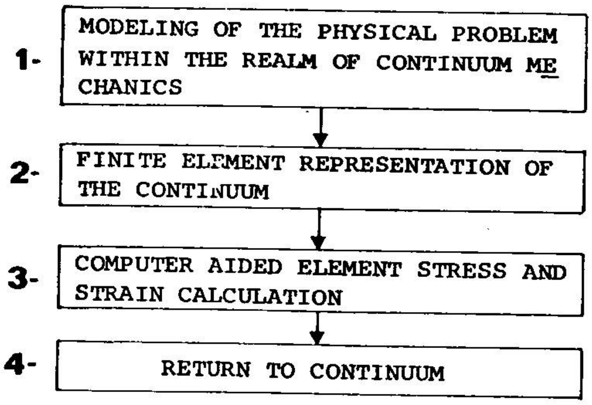


fig.1.

by which finite elements may be used for interpreting what is in reality a continuum - for us, reinforced concrete. The present au thors find it strange that, in a field in which so much importance is rightly given to reliability, so little interest is placed in researching the suitable finite element models for concrete.

It has even been suggested that certain structural problems could be dealt with by using two different computer programs for the finite element approach, so that one would be a check on the other. It may be true that practical experience well applied can be of great help in processing data, but it is equally true that two uncertainties do not make one certainty.

The present authors feel that, after all, knowledge in this field being what it is at the moment, the basic study for a nuclear building must be carried out with reference to the real continuous system, working, when necessary, on ranges of values attributed to the paramenters involved, so as to form an overall picture of the structural response.

Along with this approach, a numerical analysis by the finite element approach would also be suitable, carried out with referen ce to a restricted group of loading conditions already singled out as the most severe to be expected, and applied to those parts of the structure where the continuous structural arrangement adopted may really prove defective: interface areas between different structural elements, sudden changes in thickness, load concentra tions etc.

The last point to be considered will be the methods for check ing the calculations, both for the total structure and for the details.

In the first, overall case, a general analysis by finite elements could be used, with reference to a few carefully chosen load ing conditions, or else experiments could be made on a model. In the case of details, test models could be used of those critical areas that had previously been analysed by finite elements (see fig.2).

Coming back to the four blocks shown in fig.1 which summarise the necessary processes for structural analysis by finite elements, it is worth pointing out the marked but little known anisotropy of reinforced concrete when subject to two and three-dimensional stresses.

So, for example, in a foundation mat subdivided, along its thickness, into five layers, the upper and lower layers would be heavily reinforced, much less so the three internal layers. How can this be accounted for in the calculations?

Or again, in areas subject to tensile stress, microcraking may arise, thus altering the ways in which the structural element might deform. How can this be allowed for?

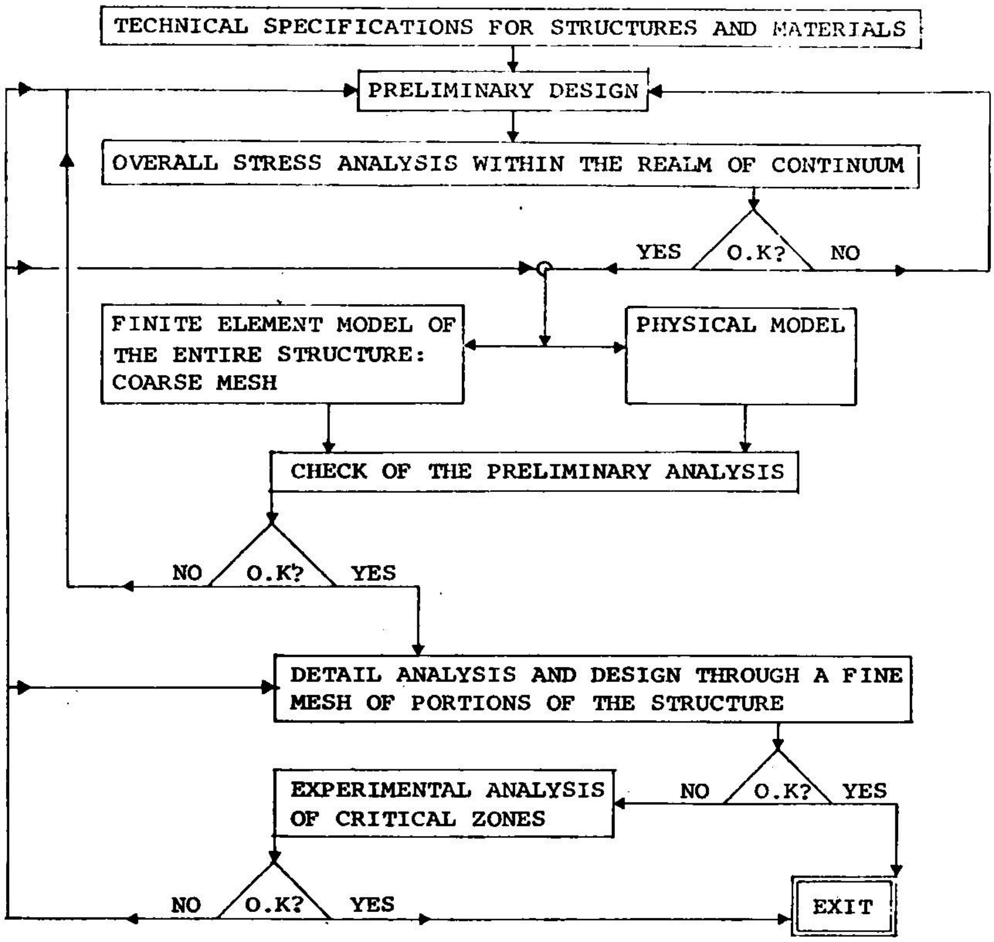


fig.2.

Looked at in this way, the problem would pose unanswerable questions and, purely from the point of view of control calculations, no solution would be possible. Something has to be done about the mathematical layout of the calculation model. That is to say, the evaluation of the stiffness must be in harmony with the design criteria for the reinforcing, i.e. concrete has to be considered as if cracked.

However, this is no simple matter, since it implies a study of all loading combinations plus an iterative examination of mathematical models according to the state of stress. Furthermore, it means that at least biaxial cracking must be allowed for, and, at least so far as the present authors know, this is still an open question, both theoretically and experimentally. There have been many studies, some quite recent, on the domains of resistance [1,2,3,4] , and on how unreinforced concrete behaves when cracked [5,6,7,8] , but not much is known about the distribution and width of cracks in reinforced concrete. Furthermore, it seems of doubtful value to make unconditional assumptions, for calculation purposes, about structural rigidities under the hypothesis of cracked concrete, without being able to estimate dangers (leakage, corrosion) that can be quantified only when more is known about the width and spacing of the cracks. Designers and licensing authorities must therefore examine the problem together in order to base simplified calculation and checking methods on what real experience there is in this field.

3. THERMAL EFFECTS

This section will be concerned with three main arguments:

1) the heat transfer law across a massive concrete wall;
2) the elastic properties of the concrete related to changes in temperature;
3) the stress analysis.

As to the first topic, consider an infinite wall of thickness d, initially at a uniform temperature. When it is exposed to cooler air on both faces, the heat transfer from concrete to the air up to the instant t can be expressed, under certain assumptions,

1) $$\Delta Q = Q_0 \exp\left(-\frac{k}{d^2} t\right)$$

where k is a suitable constant depending on the heat capacity and conductivity of the concrete. Eq.1. says that the time constant for the temperature variation is inversely proportional to the square of the thickness d . For instance for a 15 cm thick wall, 9..o of the heat in the concrete will be last to the air in 1,5 h. For a 1 m thick wall this amount of heat would be lost in 3 days and for a 1.5 m thick wall in a week. Another implication of the above assumption is that at the surface of a thick exterior wall the temperature of the concrete follows almost completely the daily variations of air temperature, while 60 cm from the surface only 10% of the daily surface temperature is felt in the concrete.

The consequences of such thermal behavior are twofold:

1) in a thick external wall the surface is often defenceless against stress cracking,
2) the structural stresses due to the average temperature rise of the entire cross section are to be related to the air temperature averaged over a few days or a week.

In fact, consider a temperature variation of 25°C acting so fast on the surface of a thick wall that the interior of the wall could be thought of as insensitive to it.

For a concrete with 300.000 Kg/cm^2 elastic modulus E, without cracking, the surface stresses would vary to about 75 Kg/cm^2 in both horizontal and vertical directions.The concrete can quite easily take such a stress wehn in compression, but cannot be relied on when in tension (in the horizontal direction assume no stress for dead load). In this latter case cracking is therefore nearly unavoidable; it is only possible to control the width and the distance of fissures by means of a suitable disposal of reinforcing.

On the other hand the overall behavior of the structure is only related to the average temperature rise of the entire cross section, which shows a smaller temperature change in both cases 1) and 2), depending on the heat transfer law.

As to leakage control, in the above picture the surface cracking due to temperature changes is confined to a relatively thin superficial region. That is, the surface, even cracked, protects the structural integrity of the concrete below it.

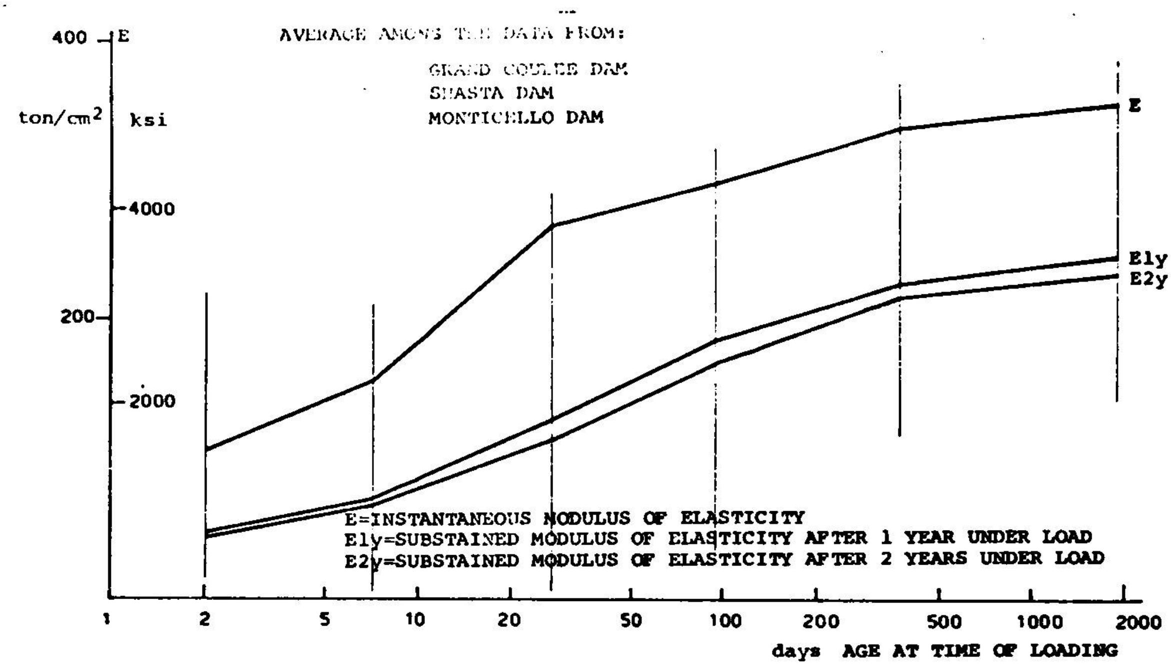
The "thermal shock" just mentioned may represent the effect of an accident of any sort or a noticeable daily excursion to which the interior concrete of a thick wall cannot take up.

Apart from the accident conditions which are confined to the reactor building, the seasonal temperature cycles only affect the state of stress of a structure very slowly provided that its exterior walls are thick enough to make them sensitive to the mean weekly variations only. Now as is well known, the elastic modulus of normal elasticity depends on the time history of the loads:instantaneous loading are related to a value of which is nearly the double of the value to be accounted for where the loading history is accomplished in some months, see for instances the Figs. 3 and 4. The data there reported refer to arch dams where similar and more severe problems are encountered.

As to figs. 3 and 4 it may be noticed:

1) according to the available data, even after 5 years, the concrete offers greater rigidity when acted on by instantaneous loading, although the ratio between instantaneous and substained rigidity decreases as the concrete ages;
2) there is evidence of a general trend toward the increase of the modulus as the concrete ages.

The first of these observations suggests that the seasonal effects on a thick walled structure are to be taken into account with a reduced modulus of elasticity, all along the structure life. On the other hand, according to the second consideration,every effect shares a different elastic behaviour for different structural age, in particular temperature cycles produce more severe


AVERAGE AMONG THE DATA FROM:
GRAND COULEE DAM
SHASTA DAM
MONTICELLO DAM
400
200
E
ton/cm2
ksi
4000
2000
E
E1y
E2y
E=INSTANTANEOUS MODULUS OF ELASTICITY
E1y=SUBSTAINED MODULUS OF ELASTICITY AFTER 1 YEAR UNDER LOAD
E2y=SUBSTAINED MODULUS OF ELASTICITY AFTER 2 YEARS UNDER LOAD
1
2
5
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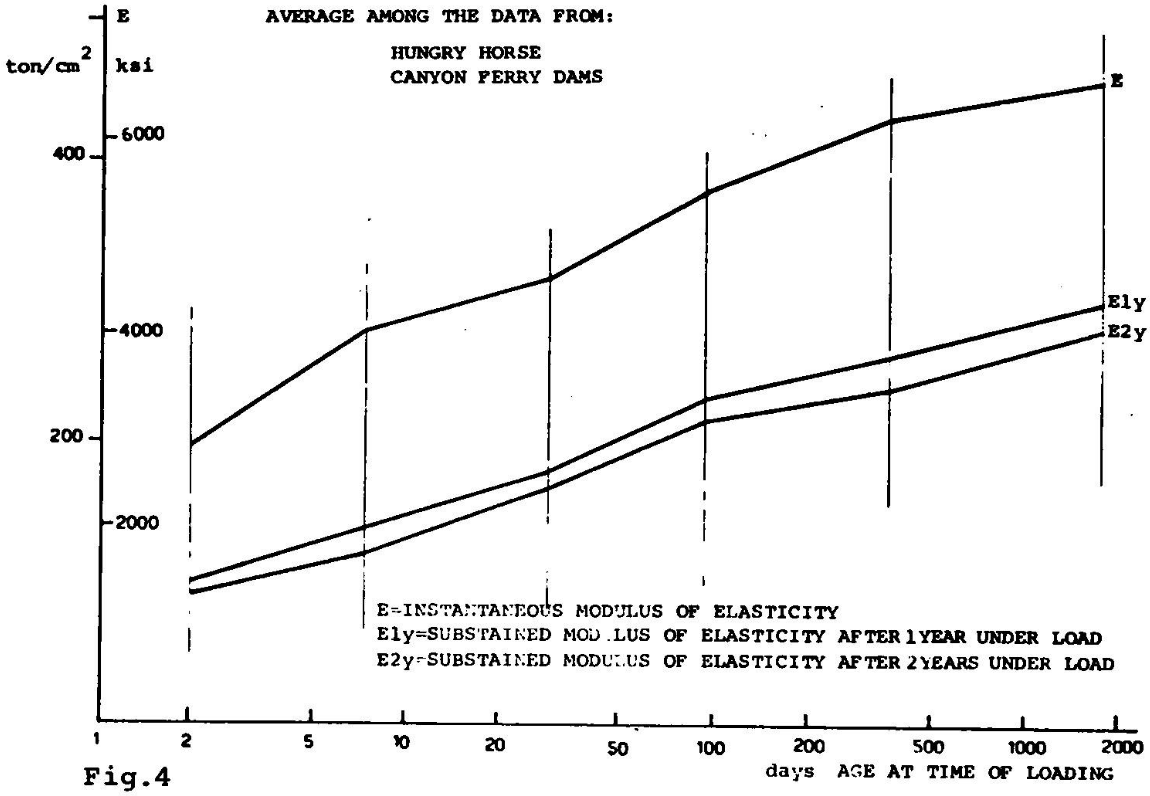
50
100
200
500
1000
2000
days AGE AT TIME OF LOADING

Fig.3


AVERAGE AMONG THE DATA FROM:
HUNGRY HORSE
CANYON FERRY DAMS
E
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6000
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E2y
E=INSTANTANEOUS MODULUS OF ELASTICITY
E1y=SUBSTAINED MODULUS OF ELASTICITY AFTER 1 YEAR UNDER LOAD
E2y=SUBSTAINED MODULUS OF ELASTICITY AFTER 2 YEARS UNDER LOAD
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Fig.4

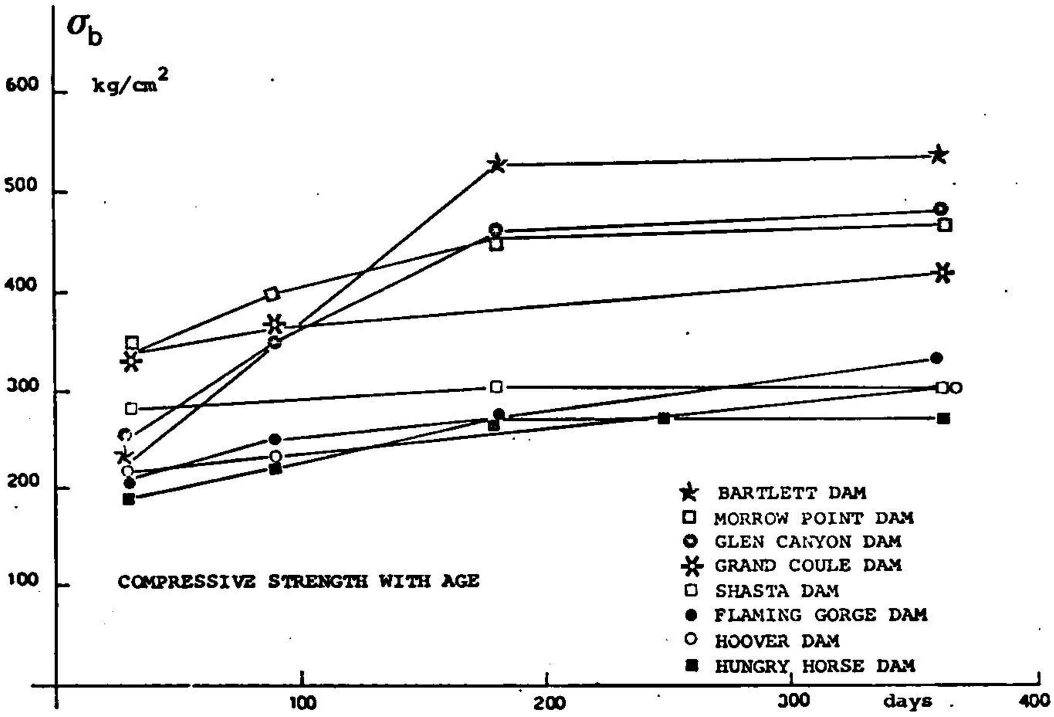

fig.5

effects when the elastic moduli are increased.

As to the safety analysis it may be further mentioned:

1) the strength of the concrete also increases with age, see for instance fig.5, (the shear strength shows similar tendencies);
2) the stresses due to applied loads, to be superimposed on temperature stresses, are not affected by a uniform increase of structural rigidity, so that, in theory, a variety of the loading combinations ought to be considered, depending on the age of the concrete.

As far as loading combinations and the relevant stress analyses are concerned, the lack of something equivalent to the ASME 3 code - Nuclear Vessel - is evidente. In fact, the ACI Committee 349, Code - Design Criteria for Reinf. Concrete Nuclear Reaction Cont. Structures - allows a nonlinear approach in the stress evaluation, which means that one can rely on the material ductility., although this occurs in the greatest portion of cases with the formation of crackings. But ACI Committee 349 does not allow peak stresses to be disregarded, as ASME 3 Code does, so that, in theory it is possible to rely on ductility provided that one is able to show in each case the relevancy of cracking.

It must be further mentioned that the stress release due to section partialization is to be taken into account, for the sake of coherence; in fact this is the physical model for any reinforcement design.

In particular, the conventional elastic analysis technique is not directly suited to application in a containment structure, where severe strain effects at discontinuities lead to cracking.

4. DISCONTINUITY EFFECTS

A typical discontinuity effect occurs where axial expansion of a slab produces large local bending and circumferential traction in a cylindrical containment structure.

For the containment structure of Fig.6, let p be the undeter-

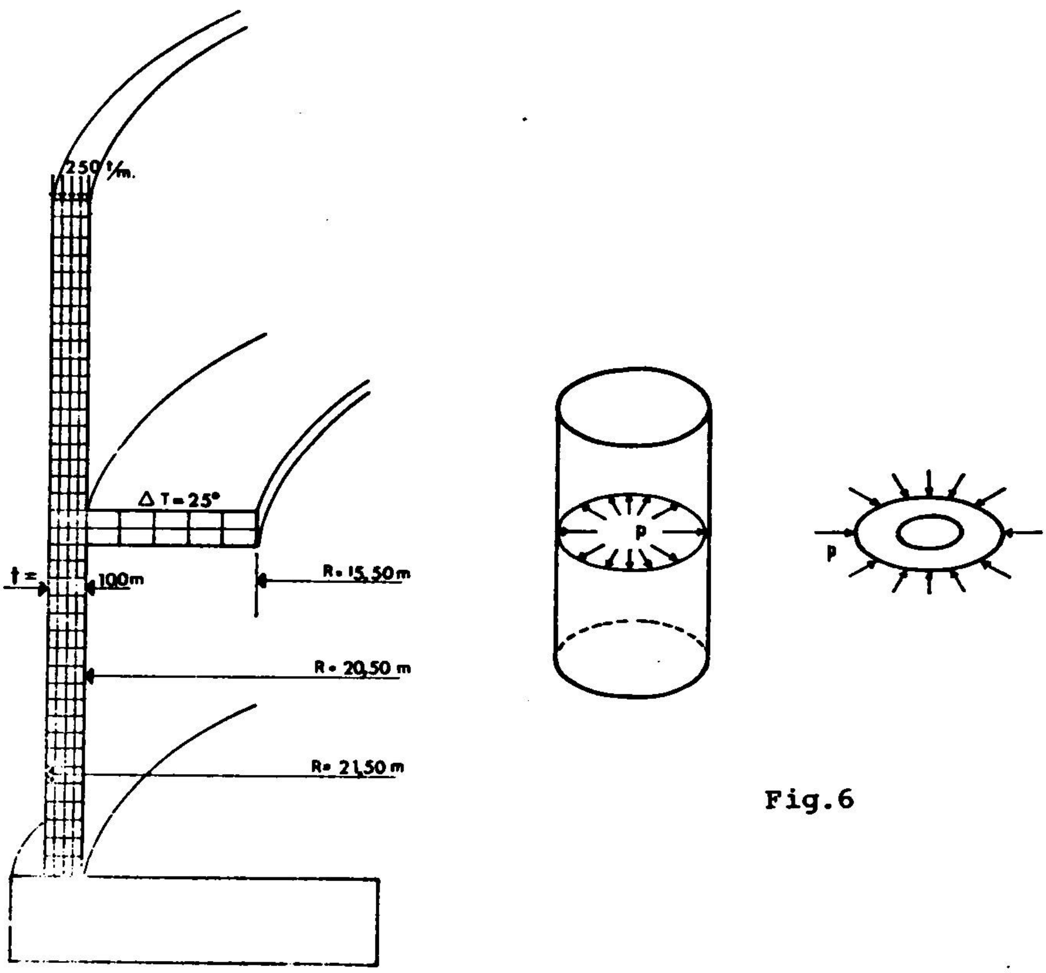

Fig.6

minate action exchanged between the slab and the shell. As a first approximation, the radial displacement produced by p on the slab may be disregarded, in comparison to the same quantity for the shell, i.e., the slab radial expansion may be taken as $s_r = \alpha \Delta T \cdot R$

Under this hypothesis, according to the thin elastic shell theory: $p = \frac{2\alpha \Delta T}{R\lambda} E t$

and the shell longitudinal moment is:

$$M_{\ell} = \frac{p}{4\lambda} \left[e^{-\lambda x} (\cos \lambda x + \sin \lambda x) \right] - \frac{p}{2} \left[\frac{1}{\lambda} e^{-\lambda x} \sin \lambda x \right]$$

where:

$$\lambda \simeq 1{,}3 / \sqrt{Rt}$$

The maximum value of M_{ℓ} is the well known expression:

$$\overline{M}_{\ell} = 0{,}294\, \alpha \Delta T E t^2$$

In the same way the following maximum values are computed:

$$\overline{M}_c = \nu \overline{M}_{\ell}$$

$$\overline{N}_c = \alpha \Delta T E t$$

Such a procedure leads generally to unacceptable designs even if a reduced modulus of elasticity has been used to account for creep releasing. The suitability of a nonlinear analysis is therefore evident.

The plastic analysis of shells by finite element methods have been extensively tried out [9,10] . The procedure is also applied in commercially available computer codes (Nastran,Feast 3, Poco..)

As to tridimensional reinforced concrete structures, the procedures so far quoted and the relevant codes are not directly applicable owing to the brittle nature of the tensile behaviour of the concrete. A few models for such a behaviour have been proposed [11,12,13] . In view of the smoothed stress pattern in the considered example, such models may be substantially simplified.

Without going into detail, the aim is to produce, in the final step, a mesh in which, for any element, the rigidities both circumferential and longitudinal are consistent with the stress levels. A shear transfer across cracks is allowed, other than the dowel action provided by the reinforcement, according to some recent investigations [14] .

The main results - see Fig.7 - are as follows.

- Circumferential actions N_c, which are to be feared in that fissures are not closed by dead load, are substantially reduced by the appropriate analysis. This is due to the fact that small

	thin shell elastic teory [Eq.1]	elastic analysis	inelastic analysis
N_c [t/m]	500	183	51
M_c [tm/m]	29.4	34.5	4.2
S [mm]	5.1	2.53	3.8

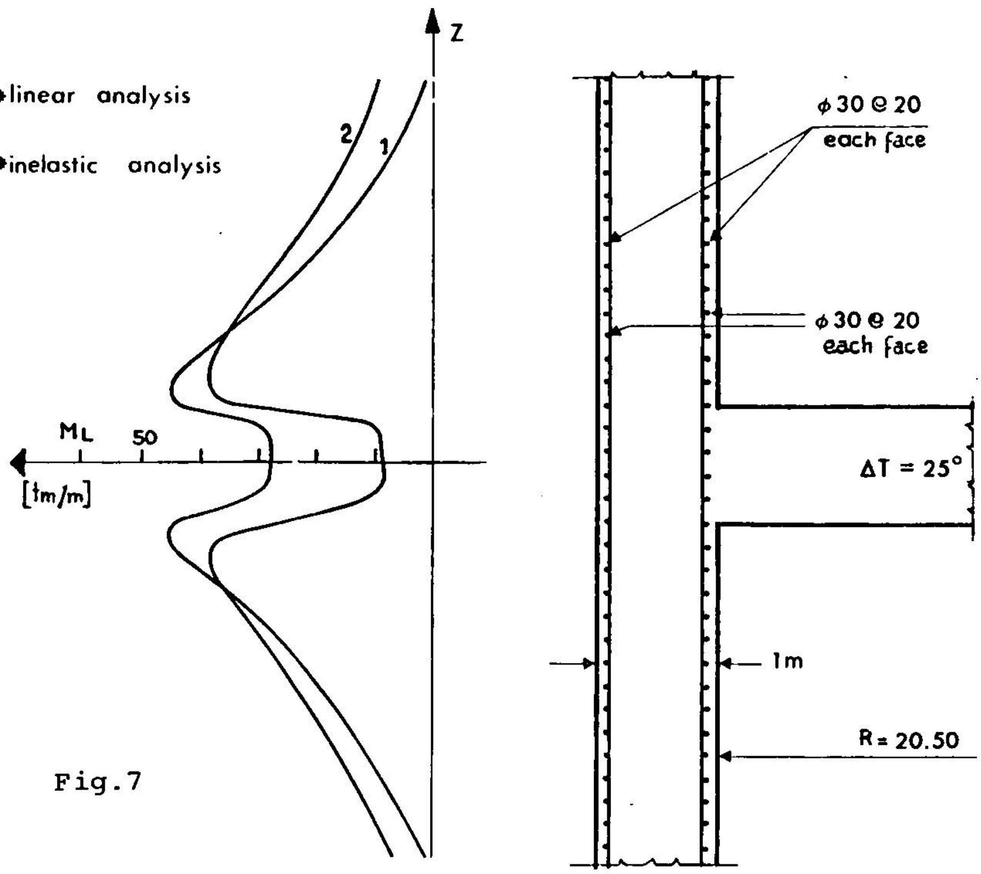

Fig.7

inelastic strains, but all along the circumference and throughout the entire shell thickness, are allowed. The same happens for circumferential moment M_c.

- On the contrary an elastic analysis gives values of longitudinal moments M_ℓ not far from those obtained through a non linear approach. May also be that the latter are greater than the former, if the shell radial stiffness is not meaningless compared to the slab.This holds obviously if one takes into account the same mesh both for the elastic and inelastic approach.

- On the contrary if the thin shell theory is directly used, a peak value for the longitudinal moments M_ℓ appears which is far greater than reality.

In conclusion the above example points out that a non linear approach gives vanishing values for M_c and N_c while proper values for M_ℓ can be obtained through a linear analysis provided that the model is not based on the thin shell theory when this is not suitable.

5 - TURBO PEDESTAL

The peculiar limit state to which this kind of structure is referred is the deformation under dynamic loading due to machine vibrations. While the pertinent level of stresses is generally fairly low, deformations are source of heating and wearing on the bearing supports and finally of machine misperformances.

The conventional approach is based on the analysis of the first three vibration modes or, which is equivalent, on the analysis of the sidewise lateral rigidity see fig.8. The fair behaviour of small and medium sized pedestals is at the basis of such an approach. In fact for these structures higher modes of vibrations, others than those considered are seldom excited. So, disregarding them is allowed, even if deformation of the shaft and the relevant poorperformances mainly influence the higher modes.

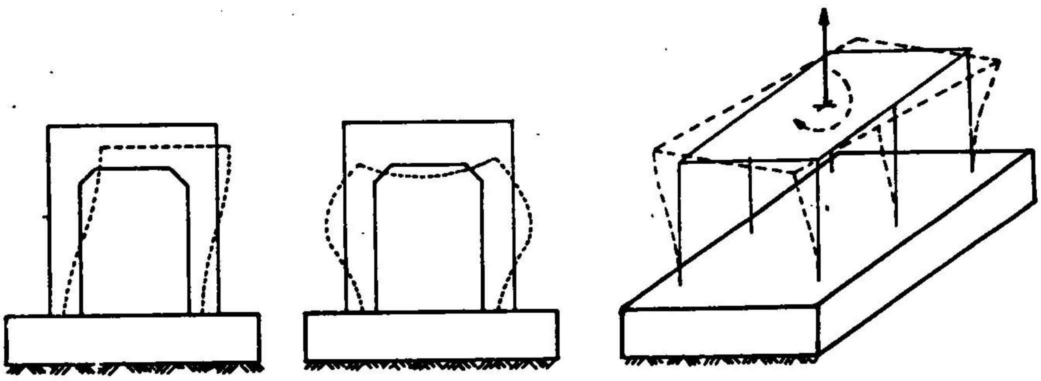

fig.8.

When the dimensions of the pedestals are increased, up to 40 60 m in length, the structure is generally undertuned in its overall behaviour. On the other hand higher modes become critical, which implies that the conventional approach is not sufficient.

The analysis for the behaviour at higher modes of vibration faces several difficulties.

- If the analysis shows that the higher modes are undertuned for the initial value of the modulus of elasticity, this may no longer be true as the concrete ages, see Fig.3. So that the only reliable situation is the one in which all higher modes, other than those in fig.8, are overtuned.
- Shrinkage and temperature stresses, both for seasonal or functional reasons, are quite important due to the large dimensions of the structures. Unless a strictly elastic analysis is satisfied, the structure may differ from the mathematical model on which dynamical analysis is based. An elastic verification of shrinkage and temperature stresses allowing a suitable tensile strength for the concrete is therefore advisable. On the other hand, an inelastic analysis based on stress release due to concrete cracking, leads to meaningless stresses for such structures.
- The shape of the pedestal pertains more to bi or tridimensional structures than to a frame structure, so that the finite element approach or a physical model approach is compulsory. It must be mentioned in this regard that the Italian National Council for Electric Energy - ENEL - has for several years tested its plants with such techniques, mainly by means of the ISMES facilities.
- The rigidity of the machine itself contributes to the structural rigidity of the working slab. There is a lack of experimental evidence in this regard.

An advisable design would make the structure as rigid as possible in order to have it overtuned for higher modes even for the initial value of the modulus of elasticity. Thus the overtuning will hold all through its structural life provided that shrinkage and thermal effects are properly arranged. This tendency has led in some G.E. plants to linking the foundation mat to the turbine building mat. The foundation deformation, in fact, is proved to contribute significantly to pedestal rigidity, see Fig.9. A further progression of this tendency is given by an integrated system where the turbine pedestal is rigidly connected to the surrounding building at every level. A bearing wall, box system structure fulfils this arrangement. A very accurate lay out of the plant must be devised before hand for this kind of structure.

6. DETAILING

In one-dimensional structures normally the state of stress alone conditions the dimensions and geometrical layout of the reinforcing. In massive concrete constructions the reinforcing spatial mesh (dimensions and geometrical layout) is instead heavily conditioned by design choices of a technical kind, such as:

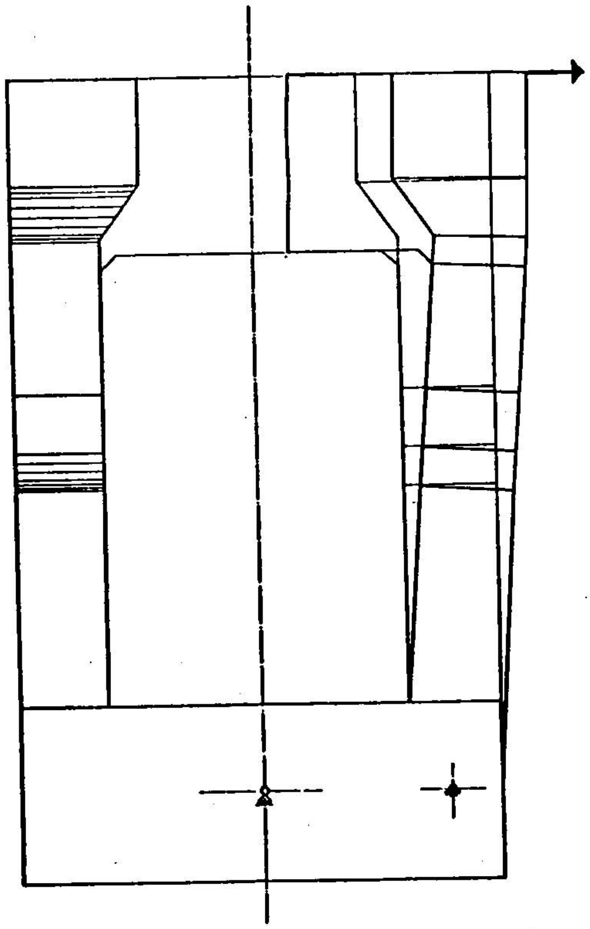

fig. 9

- the real possibilities of effectively arranging the steel reinforcing
- the practicability of casting, and the possibility of correct and adequate vibration.

These reservations dictated by the need to ensure correct erection,often become actual conditions of feasibility.Even if this cannot yet be proved theoretically,the present authors feel that a structure with a slightly undersized reinforcement,but correctly cast and erected,is preferable to one that has formal agreement with the verifications,but with a congestion of reinforcing that would jeopardize the quality of the casting and the real possibilities for checking it.

Some attempt will now be made to clarify those problems that are inherent in the designing and geometrical layout of the reinforcing, with reference to two typical examples: foundations mats and containment structures.

6.1. FOUNDATION MATS

Foundation mats are generally rectangular or circular,cover a wide area, and are from 2 to 4 meters thick.From the static point of view, a severe condition is generally that of earthquake which would partialise the supporting soil, thus subjecting the mat to considerable bending moments and shear forces. Furthermore, axial forces may arise at the same time,due to temperature changes, shrinkage and, in the case of containment structures, accident pressures.

However, the basic state of stress is typical that of bent slabs.

To a large extent, practical technology conditions the geometrical arrangement of the reinforcing.

In rectangular mats, it seems natural to make the main reinforcing with bars arranged parallel to the sides, and in several layers. No great difficulties arise here.

Circular mats,on the other hand, may have radial-circumferencial layout of reinforcing.This seems an obvious choice,but leads to serious problems concerning the distance between the bars,since this will not only affect the flexural behaviour of the mat, but may lead to congestion near the centre.As a result,it may well be a good idea to use an orthogonal cartesian mesh instead of a polar mesh.

This is normally done for the lower reinforcing of the mat, but is not suitable for the top, as it would militate against the axial symmetry of the dowels for the containment structure.

As to the design of the reinforcing, let us consider that the analysis provide, in the area to be reinforced, as a result of loading combinations, a triaxial state of stress characterised by high values for the main stress tensions.

How does this help to design the reinforcing?

Not enough information is available on the strength of reinforced concrete since, so far as the authors know, research has been mainly directed towards states of bi and triaxial compression characteristic of structures in prestressed concrete. There seems to be only one way out: use the calculation criteria valid for one-dimensional inflected structures, deduce the bending and shear forces acting on the whole section in two directions at right angles to each other (cartesian or polar), and calculate separately the tractions in both directions at the level of the reinforcing bars. Then add them vectorialy and resolve in the two components corresponding to the mesh fixed beforehand.

The same can also be said for absorbing shear forces which may often be too strong for the concrete, in that it is subject at the same time to normal tension stresses, even if they are not very great.

All this goes to show that, at the moment, there is a considerable qualitative and conceptual gap between analysis by finite element programs and the empirical and over-simplified control criteria.

When one passes to a technological examination of what has been done so far, two points stand out.

- Most of the reinforcing is arranged in the upper and lower layers of the mat, some metres apart.
- The dimensions of the foundations call for casting several thousand of cubic metres of concrete.

Plans for casting therefore constitute a problem that cannot be ignored. There seem to be two possible solutions: cast the mat has a series of horizontal layers, or make it up in full depth blocks following a ground-plan division.

The first solution has the advantage that it reduces to a minimum the heat arising from dehydratation,and also helps in laying out the upper level of reinforcing, which could rest on the layer below. On the other hand it clashes with site practice which tries to avoid interference between casting and the positioning of the reinforcing, and above all increases the problems,often not easily quantifiable, of differential shrinkage between one layer and the next. The second solution offers advantages both in terms of statics and completion times, but implies real loadbearing metal structures that have to support the upper layers of the reinforcing, and will be lost in the casting (fig.10).

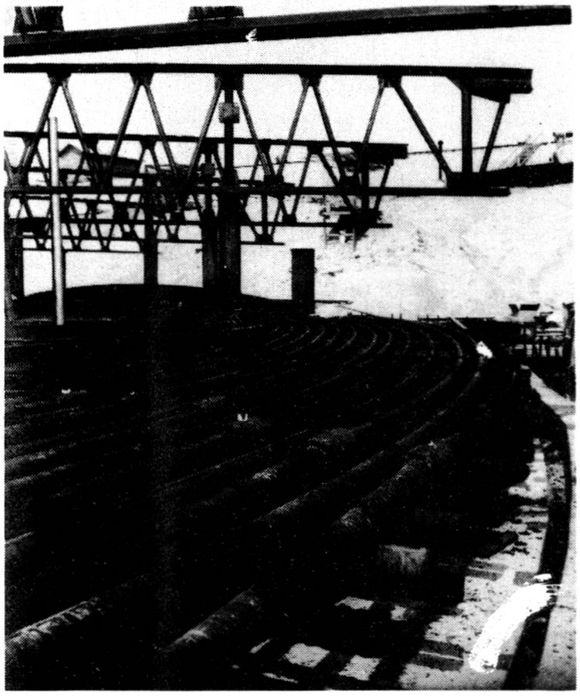

fig. 10

Furthermore,the block solution leads to greater heating in the concrete during setting, and so calls for particular expedients, such as the preventive cooling of the aggregate, or even a real refrigeration plant, that will inevitably be lost in the casting.

These problems,which could well seem even more serious and limiting than the real calculation of the foundation, make it worthwhile to ask whether there may not be solutions other than the massive slab.

In fact, a box structure of greater dimensions but reduced mass could be used, provided that a close study was made of the differential shrinkage between upper and lower slabs, as well as the distribution of concentrated loads.There is, however, another technique, recently tried out, that is perhaps even more interesting. This exploits the contribution of the metal structures needed to support the steel rods or at least makes the most of the co-operation between a steel space frame and the concrete casting of mat. In other words, and from the point of view of statics, the designer could make the best of all that was to be learned from the various stages and requirements of erection, in order to design a real metal space structure reinforced by bars embedded in the casting with both local and integrating functions.

There would certainly be great economic advantages, a reduction in completion time, and a simpler static structure without those uncertainties that arise when concrete is subjected to shear forces and at the same time to tension and bending. This is a modern solution, that has recently been successfuly tried out in works of some importance [15] .

6.2. CONTAINMENT STRUCTURES

Containment structures in reinforced concrete generally take the form of a cylinder or a truncated cone. It follows that here too the form conditions the geometrical layout of the reinforcing. This should be arranged according to the directrices and generatrices (circumferential and longitudinal reinforcing). Furthermore it will often be necessary to provide for a double helical space mesh suited to absorbing shear forces in the membrane, especially in earthquake zones. This is because the containment structure must be checked for an accompaning high internal pressure leading to a basic state of tensile stress both longitudinally and circumferencially. In such conditions the formation of efficient diagonal rods under compression becomes doubtful. The membrane stresses could be easily checked statically. Great problems, however, would arise when studying secondary effects corresponding to points where there was no continuity or where there were openings, since here the state of stress would once again have to be studied in three-dimensional terms, with all those uncertainties and difficulties already listed for mats. In these areas there would also be the risk of such a congestion of reinforcing that successful casting would be jeopardized. The basic cause of this congestion lies in having to provide adequate reinforcing against longitudinal and circumferencial bendig and radial shear (i.e. related to the thickness). This is particularly important for the openings, where the main reinforcing, arranged so as to absorb membrane stresses, has already been deviated. This is also true for the connection between the containment structure and the foundation mat.

The designer is thus obliged to make the best to provide all the reinforcing resulting from calculations. It would be useless to try to increase its thickness locally, since this would mean adding to the local rigidity, which could imply a higher value for the internal actions, which would in turn call for more reinforcing.

All of this, added to the ideas expressed at the end of section 4, leads the authors to suggest the following concept for designing the reinforcement. Calculate:

a) the internal membrane forces on models that take into account noncracked concrete;
b) the internal membrane forces on models that take into account cracked concrete;
c) secondary internal forces, bending and shear, on the same models as in b).

The dimensions of the reinforcing will follow the normal theory for reinforced concrete in the most unfavourable combination between (a+c) and (b+c). The result would certainly be a balanced situation in which congruence would be satisfied at the price of cracking due only to those internal forces that originate in the congruence itself.

7. CONCLUSION

The major items to be further developed in the future for improving the design and erection of reinforced concrete structures may be summarised as follows:

- A reduction in the conceptual gap between calculating the state of stress and designing the structure
- Interchange of ideas between structural and mechanical engineers in order to reach a more realistic evaluation of leakage in the event of craks in concrete, and to arrive at a more rational arrangement for the piping.
- A combined study, by both designers and licensing authorities, to find more realistic calculation models.
- An analysis up to define the limits of reinforcement congestion.
- A study of those solutions that allow for the use of prefabricated steel frames that co-operate with the concrete in order to reduce the problems of placing the reinforcement.

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SUMMARY

The Authors, as involved in the structural design of some recent Italian Nuclear Plants, point out the following items that seem to be widely open to discussion:

- Gap between threedimensional elastic stress analysis and design of reinforced concrete structures.
- Interaction between microcracking and stresses due to loading conditions when combined with temperature effects.
- Interface problems between parts of a massive structure analysed through different mathematical models.
- Liability of the finite element approach for dynamic analysis of turbo-pedestals.
- Congestion of rebars and connected problems of detailing and pouring.

ZUSAMMENFASSUNG

Die Autoren die mit dem strukturellen Projekt von eigigen neuen italienischen nukleare Anlagen beschaftigt sind, machen auf folgende Punkte aufruerksam die offen zur Diskussion stehen:

- Raum zwischen dreidimensionalen analytischen elastischen Belastungen und Projekt von verstarkten konkreten Strukturen.
- Zwischenaktion zwischen Mikrocracking und Belastungen verursacht durch Ladungskonditionen wenn sie mit Temperatureffekten kombiniert sind.
- Probleme zwischen Teilen einer massiver Struktur, die durch verschiedene mathematische Modelle analysiert wurden.
- Annaherender Spielraum der endlichen Elemente fur die dynamische Analysen der Turbo-Sockel.
- Inanspruchnahme der Armaturen und daraus folgendenProblemen der Detaillierung und des Betonieren.

RESUME

Les Auteurs, étant engagés dans le projet structural de plusieurs récentes installations nucléaires italiennes font remarquer que les points ci dessous restent ouverts à une ample discussion:

- lacune entre l'analyse élastique à trois dimensions de l'effort et le projet de structures en béton armé.
- Interaction entre microcracking et efforts dus aux conditions de chargement lorsque combinés avec des effets de température.
- Problèmes de liaison entre les parties d'une structure massive analysés à l'aide de différents modèles mathématiques.
- Crédibilité de l'approche à l'aide d'éléments finis pour l'analyse dynamique de turbo-piédestaux.
- Congestion des armatures et relatifs problemes de détail et de coulage.