ADJUSTMENT OF THE FRENCH CONCRETE PAVEMENT DESIGN METHOD

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In France, concrete pavements are designed by a rational approach based on a theoretical model of the structures and on the fitting of the results of the theoretical model to observations on pavements. The method originally developed in 1988 for the Catalogue of standard new pavement structures of the Direction des Routes [1] was the object in 1994 of checks and adjustments with a view to the updating of this document. This article presents the monitoring procedure. It is based on:

- the conclusions drawn from experiments performed on the LCPC's fatigue test track between 1991 and 1993 [2,3],
- assessment of the behaviour, over a score of years, of the concrete pavements of national highways and motorways,
- theoretical modelling of some structures by the finite-elements method, using the César-Lcpc software package.

1. REMINDER OF THE FEATURES OF THE ORIGINAL DESIGN METHOD (1988)

The method used in 1988 for the routine design of pavements with concrete slabs on stabilized subbases treated each layer as an elastic continuum. The existence of discontinuities and the effect of temperature gradients (which curl the slabs, modify their bearing conditions, and so increase the loadings due to traffic) were taken into account through correction coefficients introduced in the calculation of allowable stress:

$$\sigma_{t,ad} = \sigma_t(\mathbf{NE}) * k_r * k_t * k_d * k_c$$

$\sigma_t(\mathbf{NE})$: stress at which splitting tensile failure is reached at NE loadings. **NE:** total number of 130-kN equivalent axles; $\sigma_t(\mathbf{NE}) = \sigma_6\ (\mathbf{NE}/10^6)^b$

$\mathbf{k_r}$ is a coefficient that adjusts the allowable stress value to the design risk chosen according to the factors of dispersion on the thickness (standard deviation Sh) and the results of fatigue tests (standard deviation SN): $k_r = 10^{-ub\delta}$,
u: reduced centered variable associated with the risk **r**;
b: slope of the fatigue regression line of the material; for the calculations, 1/b is taken equal to 16 for normal concrete, 15 for lean concrete, 12 for cement-stabilized sand.
δ, standard deviation combining the dispersions δ_H on the thicknesses of the layers actually applied and δ_N on the numbers of cycles to failure measured in laboratory fatigue tests:
$\delta = [SN^2 + (c^2/b^2)\ Sh^2]^{0,5}$ with, **c**: coefficient relating the stress variation to the random thickness variation Δh of the pavement, **($\log\sigma = \log\sigma_0 - c\,\Delta h$);** with common structures, **c** is of the order of **0.02 cm^{-1}.**

$\mathbf{k_t}$ is a coefficient taking account of the incidence of temperature gradients. For French climatic conditions, this coefficient is taken equal to: $k_t = 0,75$

k_d is a coefficient introduced to take account of the discontinuities of concrete pavement structures (increase of calculated stress in the middle of the slab with the continuous model); it takes the values:

* 1/1.35 for structures that are not dowelled and not continuously reinforced,
* 1/1.15 for dowelled structures,
* 1/1.10 for continuously reinforced concrete.

k_c is a fitting coefficient to adjust the results of the calculation model to the observed behaviour of pavements of the same type. For concrete pavements, the value commonly used for the fitting coefficient is 1.5 when the splitting tensile strength of the concrete is used.

It will be noted that edge and thermal effects are treated as independent and additive effects, and totalled (product $k_t * k_d$), which is a gross simplification, as we shall see.

It is the criterion of the tensile stress at the base of the cement concrete slab that will determine its thickness and that of the subbase. For the subbase, there are also considerations of the minimum thickness to ensure adequate strength during the construction phase.

2. CONCLUSIONS DRAWN FROM EXPERIMENTS ON THE FATIGUE TEST TRACK

The experiments concerned the following structures:

- short cement concrete slabs without subbase (sheet 23 of the Catalogue of standard new pavement structures [1]),
- short lean concrete slabs without subbase (variant of sheet 23, for low traffic only),
- short undowelled slabs on lean concrete subbase (sheets 5 and 6),
- short dowelled slabs on lean concrete subbase (sheet 21).

Continuously reinforced concrete (CRC) was omitted from these experiments because of uncertainties about the representativeness of small-radius circular test sections for this type of reinforced structure.

The experiment was performed on two rings each divided into four sectors using the structures indicated above. The first ring, intended for the preliminary study, has undergone 1.1 million passes of the reference load (pair with 32.5 kN per wheel); the second was designed for a life of 3 million loadings, according to the design method in force.

Descriptions of the tests and the measurement principles and a first analysis have already been presented [2, 3]. We present below only the final stage of analysis, which directly concerns the verification of the design model in force.

This analysis is based on a comparison of the damage observed at the end of the tests, on each structure tested, and that predicted by the rational design model.

The observed damage was rated by a panel of experts constituted first to evaluate and quantify its evolution slab by slab and second to aggregate the various damage levels into a global level for each sector. These observed failure rates were then compared to to the theoretical damage rates, these last being identified with the risks of failure as defined in the design method.

A global evaluation of damage to the structures for 5 values of the number of axle passes is given in table 1 for ring 1 and ring 2.

2.1 - Evolution of Damage with Traffic

A clear divergence between the fatigue test track and the theoretical model is observed in the rates of evolution of damage with the number of passes. On the fatigue test track, the observed damage evolution rates are systematically very fast once the damage has begun. Comparatively, the risks of failure calculated by the Alizé model evolve in a way that is clearly slower and more gradual (figure 1).

Numerically, the rate of evolution of the risks with increasing traffic is governed by parameters b and δ (see above). Smaller values of parameters b and δ would naturally lead to faster evolution of the calculated risks.

However, the model used is based solely on the onset of damage due to fatigue of the material concrete. The findings on the fatigue test track, like those on roads and motorways, show good prediction of the onset of damage as long as it is in fact due to fatigue, without modification of the functioning of the surfacing, in particular of the bearing conditions on the subbase or base, in other words as long as it does not exceed a few percent. As soon as other phenomena modify the functioning of the slabs, (pumping, erosion, etc.), the model ceases to be representative. This is especially visible on pavements in which lateral drainage and a subbase relatively resistant to erosion significantly delay the onset of these phenomena (see below).

2.2 - Conclusion on the Fitting of the Design Method

For each structure, the procedure of adjustment between the experimental data and the results of the design method consisted of finding those values of the parameters of the model giving the best agreement between the global damage found at the end of the test (1.1 or 3.3 10^6 loadings depending on the ring) and the risks of failure indicated by the calculation. The analysis was done on two assumptions concerning the condition of the interface between the concrete and its base (slippery or perfectly bonded).

The main conclusions are the following:

- the design method suitably describes the behaviour of structures with lean concrete subbases when fitting coefficients appropriate to each structure are used. The method correctly reproduces the rank ordering, in terms of damage, of the various structures and of the various slabs of a given structure,
- on the other hand, the design method applies poorly to structures without subbases, especially in the case of the lean concrete surfacing, because of the importance in this case of pumping and erosion of the subgrade, not yet modelled,
- the model leads to a high tensile stress in the capping layers of structures with subbases, or the capping layer in other structures; this tensile stress increases when the interface between the capping layer and the subbase is bonded; this bonding was discovered when the slabs were removed at the end of the experiment. The durability of this phenomenon of bonding under substantially higher traffic, or for actual service lives including a larger number of temperature cycles, could not be assessed by the tests on the fatigue test track. The design model, by assuming a slippery interface for the whole life between the subbase and its subgrade, probably radically simplifies the true mode of operation of the structure.

3. ASSESSMENT OF THE BEHAVIOUR OF PAVEMENTS IN SERVICE

Concrete pavement structures are designed for a long initial service life (more than 20 years) with a small risk of failure during this period (less than 3 %). Except for situations where a major construction or design fault causes an early pathology, examination of the behaviour of pavements in service yields useful information for precisely judging the design only if the total traffic carried is close to or greater than that for which the pavement was designed. Information about the structures (geometry, thicknesses, etc.) and about the mechanical characteristics of the materials is also necessary. Also, the number of completed projects and the lengths of observation available vary considerably between techniques (seven cases have been analyzed and are summed up in table 2).

The analysis was done statistically, by considering in each case sections that are homogeneous in terms of pavement structure; this is justified by the probabilistic character of the design method.

It will be noted that the set of structures analyzed includes neither "thick slabs" nor continuously reinforced concrete, but only structures consisting of a cement concrete slab on a subbase of cement-treated materials or of lean concrete; the slabs are dowelled in only one case.

The reasons for not including structures consisting of slabs on a draining layer or of continuously reinforced concrete in this study are stated before the case of slabs on a treated subbase is examined in detail.

3.1 - Slab on Draining Layer (or "Thick Slab")

This kind of pavement first used in U.S., consists of a thick single plain concrete layer on draining layer. The pavement thickness combines subbase and pavement thicknesses of current plain concrete pavement. This design derives not from the rational method but from empirical considerations. The concrete pavement must be

made thick enough so that meshing ensures proper load transfer and limits the loadings on the base. For high and medium traffics, this design therefore results neither from consideration of the stresses in the concrete slab nor from a calculation of the loading of the base. Experience shows that the basic element determining the durability of these structures is the correct functioning of the draining layer; when the water in the structure is evacuated by draining layer, it is no evolution of the capping layer. When water stays in draining layer, erosion degrades the support. At the present time, after 15 years' service, at most, for the oldest sections, there has not yet been a precise assessment of the condition of these pavements on the privately-operated motorway sector.

3.2 - Continuously Reinforced Concrete

The continuously reinforced concrete solution is also a relatively recent approach in France; the first projects date from 1983, and the use of flat notched high-yield-strength steels from 1987. While CRC projects *built according to the rules of the art* exhibit no damage suggesting that current design is inadequate, the short history makes it impossible, conversely, to rule out possible over-engineering.

3.3 - Conclusions Concerning Undowelled Short Slabs

In the oldest pavements, emphasis was placed on the quality and thickness of the concrete slab. This seems not to have been the case with the subbase, the thickness of which seems rather to have been matched to the irregularities of levelling of the formation level. This is an especially harmful factor, given the importance of the localized loadings transmitted at the transverse joints.

The introduction of the extra width has eliminated edge deflections. The execution of generalized drainage at the time of construction has also proven to be a key feature in reducing pumping at the transverse joints.

On Paris-Lille, the difference between the dates of onset of pumping on undrained and drained sections is approximately 4 years (after roughly 5 years of service against 9 years, or a variation of 1 to 2 in total traffic), the pavements of this section having received no joint maintenance in 10 years.

The effectiveness of lateral drainage is a significant factor in the durability of the slow lane: this point was verified on Paris-Lille, where the absence of drainage has led to a very clear acceleration of slab failures on part of the Paris-to-Lille lanes monitored for 6 years.

3.4 - Dowelled Short Slabs on Subbase

The most significant case for which a significant history of observation is available is that of the slow lanes of Paris-Lyon and Paris-Lille in the Paris suburbs.

3.41 - Paris-Lyon, Paris suburbs

The slow lanes were rebuilt from 1983 using dowelled slabs with a structure consisting of a 28-cm concrete slab on a 15-cm subbase of lean concrete and 17 to 37 cm of treated sand.

In 1993, the total traffic exceeded 20 and 10 million trucks on the two sections, respectively. The 1992 damage survey found 3.2 and 5.6 % of the slabs cracked by fatigue. These pavements also seem to have been affected by some settlement of the lower layers, which can in turn induce cracking of the slabs. As regards the effectiveness of the dowelling, the 1992 survey found differential deflection values of 0.10 to 0.15 mm on the dowelled lane, while the deflection of the undowelled centre lanes was 0.50 to 0.80 mm. While it is true that the subbase of the truck lane is stronger, the dowelling still has some effect after 10 years and a total traffic greater than 10 million trucks (or approximately 13×10^6 standard 130-kN axles).

3.42 - Paris-Lille, Paris suburbs

As an experiment, in June 1984, dowels were placed at the transverse joints of 22 slabs in the slow lane. It was found in 1987, after a total traffic of 6.1 million trucks, that:

- the dowels had stood up well, with no failure or ovalization of their recesses,
- they affected the rate of onset of slab breakage as compared to the adjacent undowelled zones.

3.5 - Conclusions Concerning Dowelled Short Slabs

The conclusions that can be drawn from Paris-Lyon concerning design are somewhat affected by the settlement of the lower layers. Note however that in spite of this unfavourable circumstance, the pavements have borne normally the traffic for which they were designed and the dowelling has remained effective as a measure to limit differential deflections between slabs and forestall the structural damage it causes.

4. THEORETICAL MODELLING OF THE STRUCTURES BY THE FINITE ELEMENTS METHOD

Starting from what was learned in the experiments on the LCPC's fatigue test track and from observation of the behaviour of pavements in service, the complementary numerical study was centered on short slabs on treated subbases, with and without dowels.

The first diagnostic stage was performed by the usual design method, which then had to be transcended by the use of other tools making it possible, now, to suitably represent the discontinuous and finite character of the slabs of these pavements. This was done using three-dimensional models created using the César-LCPC finite-elements software package (figure 2) [4].

As an example, we present here the results concerning the edge effect and stresses of thermal origin in the case of undowelled short slabs.

4.1 - Study of Edge Effect

The coefficient $1/k_d$ allowing for the increase of stress when the load is near the edge or a transverse joint was determined, in the design method in force, by application of the solution of Westergaard's approximate model and Miner's damage accumulation law plus some assumptions concerning the opening of the joints [5]. The study was repeated here with the finite-elements method, using César-LCPC software and a three-dimensional elastic model.

The finite-elements calculations were done by selecting the geometry of the extreme structures of sheet 5 (slab of cement concrete (BC) on subbase of lean concrete (Bm)): T0, PF1 (28 BC on 18 Bm) and T0, PF3 (20 BC on 12 Bm), together with the intermediate case T2, PF2 (24 BC on 12 Bm); Ti and Pfi are defined in table 3 [6].

The effect of introducing the extra width was examined in the T0, PF1 case. With no extra width or with a small extra width, the loading in the slab is maximum when the load is halfway between the transverse joints and near the longitudinal edge. The coefficient $1/k_d$ takes a value of approximately 1.35 when the right edge of the axle is about 20 cm from the longitudinal edge of the slab.

Increasing the extra width reduces the stresses in the slab when the load travels along the longitudinal edge. With an extra width of 75 cm, the increase in stress, in the structures studied, is only about 1.08. With the extra width used (75 cm for high traffics), the worst configuration is that where the load is at the limit of the transverse joint when the latter is open. If the same weighting is applied as was adopted in 1988 to allow for the variability of the opening condition of the transverse joints in the course of the year [5], calculations by the finite-elements method indicate a stress increase coefficient ranging from 1.07 to 1.22 depending on the structure studied.

4.2 - Temperature Considerations

The value of the temperature coefficient k_t was found in calculations with a simplified reaction modulus model for the foundation mass and the use of Miner's damage accumulation law. This law was applied assuming a uniform distribution of traffic over the year and seasonal variation of the temperature gradient in the slab [5].

The value of 0.75 chosen in 1988 is a mean of 1/1.25 and 1/1.42, found in a series of calculations covering the range of variation of the values of the subgrade reaction modulus and of the slab thickness.

When the concrete slab is subjected to a temperature gradient, there is partial separation from the subbase; the problem is no longer linear because of the changes in bearing conditions. Thus the earlier superposition of the edge effect and the temperature effect is improper. The calculations done with César-LCPC software represent simultaneous application of the temperature gradient and of the load representing the half-axle. For the comparison with the weighting coefficient values chosen in 1988, the results of the finite-elements calculations must therefore be compared to the reciprocal of the value of product (k_t x k_d), which is 1.35/0.75 = 1.8.

The finite-elements calculations were done for sheet 5: cases T0, PF1 and T2, PF2, for two temperature gradient values, 0.6 and 0.3°C/cm. The transverse joint between two slabs is modelled as a free edge.

The calculations indicate that maximum loading in the slab occurs when the load is near the transverse joint. The values of the stress increase coefficient, with respect to the case of the load located at the centre of the slab, with no temperature gradient, are given in table 4.

When the load is placed at the centre of the slab, the effect of the temperature gradient leads to a stress increase, depending on the structure, of 1.4 to 1.56 for a gradient of 0.6°C/cm and 1.3 to 1.44 for 0.3°C/cm. It is obvious here that the evolution of the stresses with the value of the temperature gradient is not linear.

The stress increase coefficient to be used in the design calculation must take account of the distribution of the temperature gradient over the year. The problem is complicated by the fact that since the maximum loadings are obtained when the load is at the edge of the transverse joint, it would also be necessary to take account of the variation over the year of the load transfer between the extreme situations of joint open and joint closed. We do not yet have pertinent information to determine how to combine these two distributions (temperature gradient and joint opening) in a way that is not arbitrary. In view of the calculated maximum values, it seems that the combined effects, weighted over the year, may lead to a value of the product k_t x k_d that is slightly higher than the 1/1.8 selected for design purposes in 1988, but the difference can not be very large because of the highly nonlinear incidence of the temperature gradient.

5. CONCLUSIONS - PROPOSED NEW STRUCTURES

The analyses and assessments described below have led to adjusting the correction coefficients of the French pavement design method, published at the end of 1994 (table 5) [6].

Only cement concrete structures with joints, with and without dowels, on subbases of materials treated with hydraulic binders, including lean concrete, have been the object of systematic calculations making it possible to propose a significant modification of sheets 5 and 21 of the catalogue (table 6).

For the operating authority, this change of pavement thicknesses leads to taking a risk similar to that taken with the previous arrangements only under the following conditions:

- need to make sure of adequate drainage, especially for formation levels having the minimum bearing capacity allowed,
- need for a subbase of good quality (thickness and strength), essential with respect to the loadings in the subbase and in the subgrade.
- great utility of prior levelling of the pavement formation level, favouring better conformity of the thicknesses of the upper pavement layers.

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FIGURE 2: Example of three-dimensional modelling by César-Lcpc finite-elements software.

TABLE 1: Modelling of structures by the conventional method (Alizé).

Type of pavement	Number of passes (million)									
	0.2	**0.4**	***0.5***	**0.6**	**0.8**	***1.0***	**1.1**	***1.5***	***2.0***	***3.3***
Concrete pavement without subbase	0	0	***0***	10	25	***0***	70	***0***	***0***	***90***
	(53)	(62)	***(72)***	(66)	(70)	***(77)***	(73)	***(80)***	***(81)***	***(83)***
Lean concrete pavement without	0	0	***15***	0	35	***90***	35	***100***	***100***	***100***
subbase	(-)	(-)	***(82)***	(-)	(-)	***(83)***	(-)	***(85)***	***(85)***	***(86)***
Concrete pavement with subbase,	0	0	***0***	0	17	***0***	30	***0***	***10***	***30***
without dowels	(0)	(15)	***(12)***	(17)	(21)	***(18)***	(23)	***(22)***	***(25)***	***(31)***
Concrete pavement with subbase, with	0	0	***0***	0	0	***0***	0	***0***	***0***	***2***
dowels	(0)	(0)	***(0)***	(0)	(0)	***(1)***	(0)	***(1)***	***(2)***	***(2)***

Comparison, structure by structure, of observed damage level and calculated risks of failure (in brackets) both in percentage. Ring 1 and *Ring 2*. (-) not available.

TABLE 2 - List of Concrete Pavement Cases Analyzed

Road	Year of work	Type of work	Type of structure
Paris-Lyon, suburb	1960	new construction	short slabs
Parisienne	1983 and 1987	reconstruction	dowelled slabs
Paris - Lyon	1964 to 1976	construction	short slabs
A62	1975	construction	short slabs
Strasbourg-Basel	1978 to 1980	construction	short slabs
Paris-Lille, Paris suburbs	1976 and 1977	reconstruction	short slabs
Rennes-Saint-Brieuc	1983 and 1984	construction	short slabs

TABLE 3 - Definitions of Traffic and Formation Level Classes (Ti and PFi)

Definitions of traffic classes												
Class	T5	T4	T3-	T3+	T2-	T2+	T1-	T1+	T0-	T0+	TS-	TS+	TEX
MJA	<25	25 to 49	50 to 84	85 to 149	150 to 199	200 to 299	300 to 499	500 to 749	750 to 1199	1200 to 1999	2000 to 2999	3000 to 5000	>5000

Definition of formation level classes				
Class	PF1	PF2	PF3	PF4
Modulus (MPa)	20 to 49	50 to 119	120 to 199	200 and more

MJA: annual mean per day; the modulus of the formation level is an estimate of the long-term modulus.

TABLE 4 - Values of Stress Increase Coefficient ($1/(k_t \times k_d)$)

Structure T0,PF1	Structure T2,PF2
1.80 for a gradient of 0.6°C/cm	2.04 for a gradient of 0.6°C/cm
1.74 for a gradient of 0.3°C/cm	1.44 for a gradient of 0.3°C/cm

case of the load located at the centre of the slab with no temperature gradient

TABLE 5 - Evolution of correction coefficients between 1988 and 1994, after the adjustments of the pavement design method [6]

Structure	1988 design			1994 Technical Guide *	
	kt	reciprocal of kd	value of kt x kd	reciprocal of kd	valeur kd
Concrete surfacing with undowelled joints	0.75	1.35	0.5556	1.70	0.59
Concrete surfacing with undowelled joints	0.75	1.15	0.6522	1.47	0.68
Continuously reinforced concrete surfacing	0.75	1.1	0.682	1.47	0.68

* The Technical Guide provides only one correction coefficient

TABLE 6 - New design proposals for structures based on short cement concrete slabs (sheet 5).

	T0	T1	T2	T3
PF1	28BC+17Bm *(28+18)*	26BC+17Bm *(26+18)*	24BC+16Bm *(24+15)*	22BC+16Bm *(22+15)*
PF2	27BC+15Bm *(28+15)*	25BC+15Bm *(26+15)*	23BC+12Bm *(24+12)*	21BC+12Bm *(22+12)*
PF3	24BC+15Bm *(26+15)*	22BC+15Bm *(24+15)*	20BC+13Bm *(22+12)*	19BC+12Bm *(20+12)*

The joints are not dowelled and the subbase is made of non-eroding lean concrete. BC: surfacing concrete; Bm: subbase concrete; in brackets, thicknesses of previous sheet.

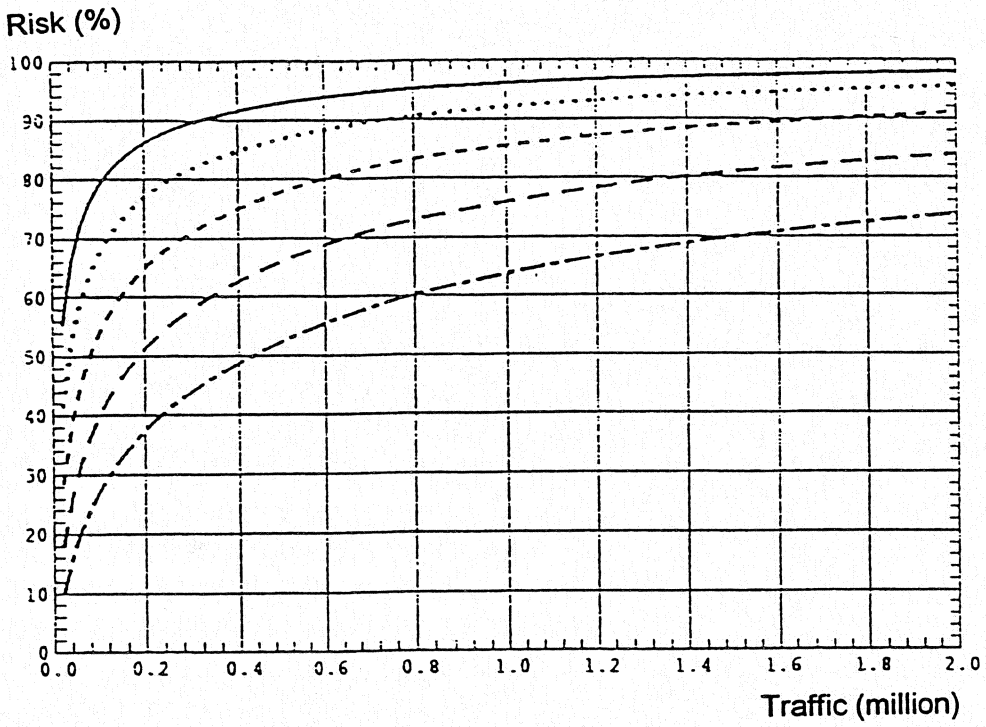


Figure 1 - Evolution of calculated risks of failure by the Alizé model (example: effect of the fatigue curve slope b).

From top to bottom, 1/b = 20, 16, 12, 8, 4

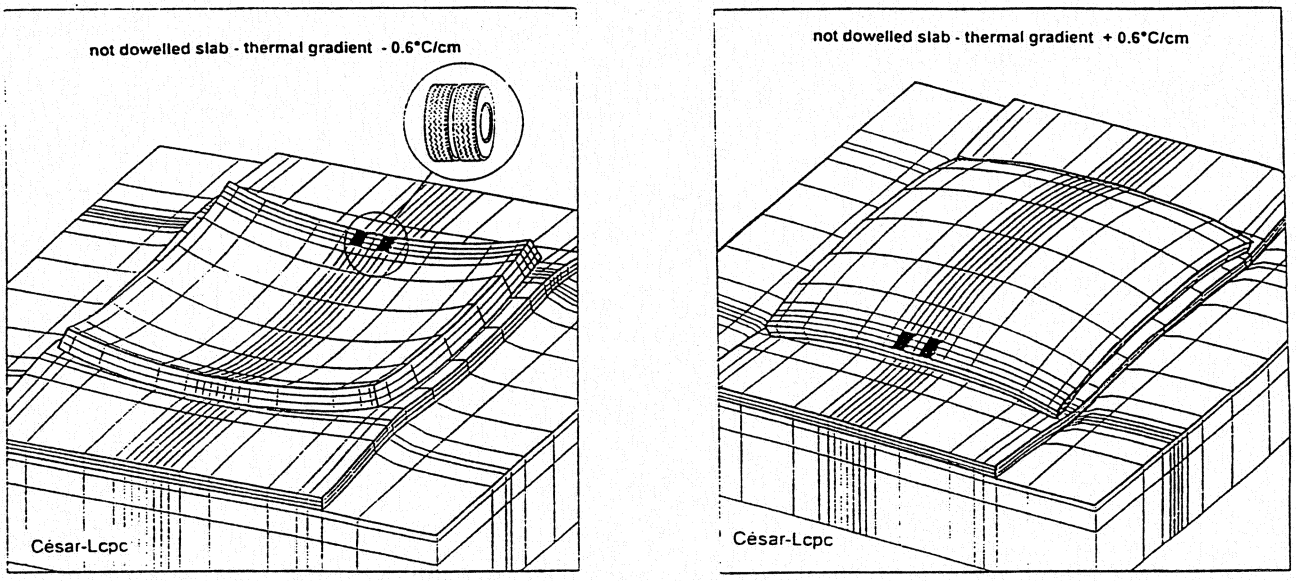


Figure 2 - Example of three-dimensional modelling by César-Lcpc finite-elements software.

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