BELGIAN EXPERIENCE WITH CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS*

Y. DeChamps
Head of Research Department of C.R.I.C.
R. DePaepe
General Director, Highways Administration, Ministry of Public Works and
P. Dutron
Director of C.R.I.C.

1. INTRODUCTION

Belgium's motorway network has been considerably extended in recent years: it comprised about 500 km (300 miles) of motorways in service at the end of 1970 and 1050 km (630 miles) at the end of 1974.

The Highways Administration very quickly realized the advantages offered by the technique of continuously reinforced concrete pavements:

- simplicity of construction and a high degree of riding comfort due to the absence of all transverse joints;
- great durability of the riding properties of the pavement under the effect of traffic loads as well as of freeze-thaw cycles and de-icing salts;
- very low maintenance costs, making continuously reinforced concrete the most economical type of pavement in the long-term view.

As far back as 1968 it was accordingly decided to apply this technique to the entire length of the Brussels - Liege and the Liege - Charleroi motorways. In March of that year a study group sponsored by the Belgian Minister of Public Works and composed of representatives of the Highways Administration and of the Research Centres of the Belgian steel and cement industries went to the United States with a view to benefiting by that country's great experience of continuously reinforced concrete pavements. Previous Belgian experience itself was already quite substantial, six experimental sections of pavement totalling 80,000 m^2 (95,680 sq yd) in area having already been built. The oldest of these, at Leuze, dates from 1950 and is still in good condition after more than 26 years of service under particularly heavy traffic.

On the basis of all this information, a major study of the subject based on the construction of three additional experimental sections enabled all the characteristics of continuously reinforced concrete motorway pavements to be firmly established. That study led us to modify the American technical methods a little to take account of certain factors peculiar to our country, including the following:

- in Belgium the maximum single axle load is 13 metric tons (28,600 lbs) as against 18,000 lbs in the United States;
- the particular composition of our highway concrete, which is well suited to our climatic conditions and which differs from American concrete mixes mainly in having a higher cement content, lower workability and the exclusion of any air-entraining agent;

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- the presence of numerous bridges and other civil engineering structures under Belgian motorways, one every 2500 m (2725 yds) on average.

At the present time Belgium has by far the largest area of continuously reinforced concrete pavements of any country in Europe.

Fig. 1 is a map showing Belgium's motorway network, indicating those sections built in continuously reinforced concrete which are already in service and those yet to be constructed by that technique. On 1 January 1975, of the total length of 1050 km (630 miles) of Belgian motorways a length of 225 km (135 miles) was paved with continuously reinforced concrete.

The design and construction of Belgian continuously reinforced concrete motorway pavements have already been the subject of many international publications, the principal ones among which are listed in Chapter 6 (refs. 1, 2, 3, 4). In Chapter 2 this paper will therefore give merely a brief description of these pavements, giving special prominence to those features which distinguish them from the American continuously reinforced concrete pavements.

This paper is more particularly concerned - in Chapter 3 - with the behaviour of our continuously reinforced concrete pavements in actual service. First of all, it can be stated that this behaviour is excellent. This is true both of the experimental pavements built between 1950 and 1970 and of the motorway pavements built on a large scale between 1971 and 1974. The behaviour of these pavements under service conditions has, however, been carefully studied and analysed so as to be able at once to suggest any modification likely to further improve the quality and durability of future pavements.

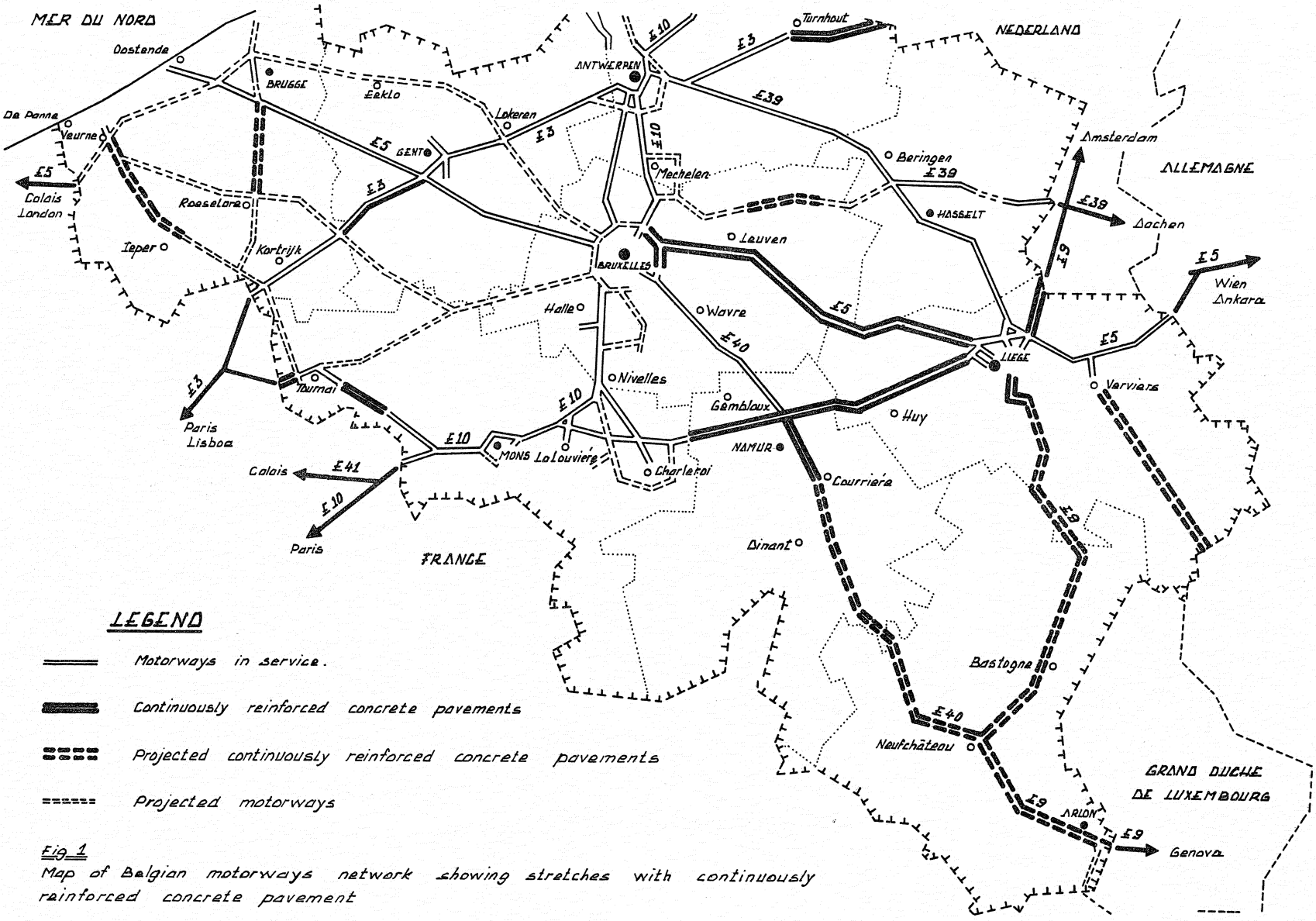
Chapter 3 comprehensively reviews and combines all the observations which have thus successively been made with regard to the cracking of the pavements, the corrosion of the reinforcement, the occurrence of after-compaction of fills, and the consequences of some defects of execution.

In the form of conlusions Chapter 4 sets forth the modifications that have already been proposed and adopted for future application on the basis of this detailed examination of the behaviour of existing pavements.

2. DESIGN AND CONSTRUCTION OF CONTINUOUSLY REINFORCED CONCRETE MOTORWAY PAVEMENTS

2.1 Standard Cross-Sections.

From 1970 onwards two typical cross-sections have been standardized for all Belgian motorway pavements, depending on the type of pavement adopted, namely, either concrete or bituminous materials. These cross-sections have been designed with reference to the traffic that they are expected to carry on motorways. Fig. 2 shows that the overall thickness of the pavement is less in the case of continuously reinforced concrete construction because of the better load distribution ensured by this type of pavement. The 6 cm (2.3 in) thick basecourse of dense bituminous mix which is always provided between the actual surfacing of continuously reinforced concrete and the lean concrete roadbase is finished to the same degree of evenness as the surface of the carriageway itself and extends for a distance of 80 cm (32 in) beyond each side of the concrete surfacing in order to provide a good working surface for the slipform pavers or for the installing of fixed road forms.



LEGEND

Motorways in service.

Continuously reinforced concrete pavements

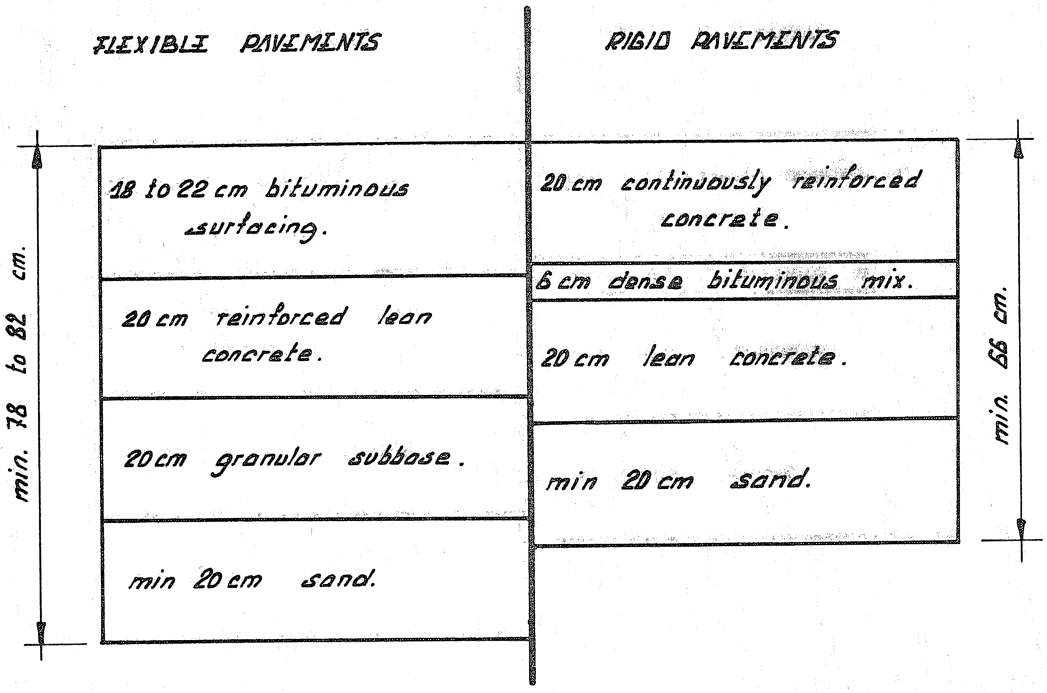
Projected continuously reinforced concrete pavements

Projected motorways

Fig 1
Map of Belgian motorways network showing stretches with continuously reinforced concrete pavement

Fig 2

CROSS-SECTION THROUGH MOTORWAY PAVEMENTS



2.2 Arrangements At Bridges And At The Ends Of The Pavement.

A terminal anchorage with six transverse anchor lugs is provided at each interruption of the continuous slab. In view of the large number of underpasses under the motorway the continuously reinforced concrete pavement is normally not interrupted over bridges of the "rigid frame" type with spans of up to 15 m (16.3 yds) or indeed up to 25 m (27.2 yds).

2.3 Reinforcement.

The very high strength of the traditional Belgian highway concrete has led to the adoption of a longitudinal reinforcement percentage of 0.85 % referred to the transverse cross-sectional area of the concrete. The yield point of the reinforcement is normally 50 kg/mm^2, but some road sections have experimentally been constructed with 0.70 % reinforcement with 60 kg/mm^2 yield point. Using higher-yield-point steel can in fact result in a saving, since the extra cost of the steel is more than offset by the reduction in the overall weight of reinforcement, which is thus reduced from 15.5 kg to 12.5 kg per m2 of pavement.
The reinforcement consists either of all-welded fabric fully made up in the factory, including the supports, or is composed of transverse bars provided with supports welded to them in the factory or workshop, the longitudinal bars subsequently being fixed to these transverse bars on the construction site by clipping the ends of the supports to them. In either case the transverse reinforcing bars are always disposed at an angle of 60° in relation to the longitudinal ones in order to counteract the tendency for cracks to develop at transverse bars and extending along them, with the attendant risk of corrosion to these small-diameter bars. The advantage of this arrangement has by now been confirmed, because more than 90 % of all transverse cracks have in fact been found to extend at right angles to the longitudinal centre-line of the pavement,

while only a very small number of cracks extend along the oblique alignment of the transverse reinforcing bars for part of their length. In addition, the oblique position of these bars makes it easier to stagger the laps in the longitudinal bars of the welded fabric reinforcement and reduces unfavourable effects on the evenness of the road surface due to vibration being set up in the reinforcement by the concreting machines.

The centre-lines of the longitudinal bars are at a depth of 7 cm ± 1 cm (2.7-.39 in) below the road surface.

For making the fabric reinforcement, a process using welding machines with preheating of the large-diameter bars had to be developed to enable 18 mm bars to be cross-welded to 12 mm (0.47 in) or 14 mm (0.55 in) bars, both bars being of high-carbon naturally hard steel.

At the same time, special rules for the quality control of the welds were established. These control checks relate to the strength of the welds in resisting transverse pull and to the preservation of the yield point and especially of the ductility of the bars at the weld (absence of embrittlement).

2.4 Concreting.

The main differences between Belgian and American highway concretes are the following :

- the Belgian concrete has a higher cement content : between 375 and 400 kg/m3 (23.5 and 25 pds/cu.ft.)

- it does not contain an air-entraining agent ;

- its workability is lower : 0 - 1 cm slump for placing between fixed forms, 3 - 5 cm slump for slipforming (0.39, 1.18 and 1.96 in)

- it has higher strength : average compressive strength measured on cylinder specimens (100 cm^2 cross-sectional area, height h = 10 cm), obtained as cores drilled from the pavement, is upwards of 750 kg/cm2 at 56 days of age.

The concrete is always placed in a single layer through the mesh of reinforcing bars pre-assembled on the base. In most cases concreting is done across a large width of 7.50 m or 11.25 to 12.25 m, corresponding to two or three traffic lanes.

The rate of concreting progress is about 35 m/hour between fixed forms and about 80 m/hour with a slipform paver.

On all the continuously reinforced concrete motorway sections constructed up to the present time the surface texture has been obtained by deep transversal grooving of fresh concrete. This treatment ensures excellent roughness, but adversely affects the road users' and the adjacent residents' comfort more particularly because it produces a high noise level. In this context it is to be noted that two other methods of ensuring a lasting and high degree of skid resistance for concrete pavements have been developed and mechanized in Belgium. These are, respectively, the chipping of fresh concrete and the aggregate exposure. As envisaged here, the operation referred to as chipping consists in inserting 16-22 mm chippings of non-polishing stone, in a quantity of 6 - 8 kg/m2, in the freshly concreted road surface and embedding them by tamping and vibration. With this method the aggregate used in the concrete itself can permissibly consist of stones that would become polished if exposed to the action of traffic. Aggregate exposure consists in laying bare the stones in the concrete at the surface of the pavement by spraying and brushing away the surface mortar uniformly from the fresh concrete as soon as it has begun to set.

2.5 Initial Cost Of Constructing Continuously Reinforced Concrete Motorway Pavements.

The average contract prices for the construction of continuously reinforced concrete pavements have not varied greatly between 1971 and 1975 and are equivalent to around 15 U.S. dollars per m2 for a 20 cm thickness of concrete surfacing, including the cost of the reinforcement. The rising cost of materials and labour over the years has been offset by the steady improvement of the construction methods and by the greater "know-how" that the contractors have gained.

For comparison, the present initial cost of constructing a 22 cm thick bituminous surfacing as envisaged in Fig. 2 is around 11 U.S. dollars per m2, but against this lower price must be set the higher cost of the road base and sub-base for such flexible pavements.

3. BEHAVIOUR OF CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS IN ACTUAL SERVICE

3.1 Survey Of Transverse Cracks.

The cracking pattern represents an important factor of the behaviour of continuously reinforced concrete pavements. Excessively wide and widely spaced cracks must evidently be avoided, but it is possible that too closely spaced cracks are also unfavourable because of the risk that certain chunks of concrete between such cracks may become unstable and detach themselves above the layer of reinforcement under the action of the horizontal forces exerted by traffic.

Since the spacing, i.e., the distance between consecutive cracks, often varies greatly about the mean value, the cracking pattern cannot be defined merely in terms of average spacing, but additionally requires establishing a complete histogram of crack spacings. The ordinates of such a diagram are the numbers of cracks in each of the cracks spacing classes, which are conventionally defined by dividing the axis of abscissae into sections increasing in steps of 20 cm, for example.

However, in order to obtain a more realistic and also a more balanced representation of the cracking pattern it is advantageous to transform these detailed histograms into diagrams in which the abscissae comprise only five classes of crack spacing and in which the ordinates are not the numbers of cracks in each of these classes but instead are the percentages of the pavement area which have crack spacings assignable to the conventionally defined crack spacing classes adopted here. A diagram of this kind provides a more realistic picture of the state of cracking of the pavement, for a given area of pavement may contain a large number of very closely spaced cracks while another area of the same size may merely contain a small number of widely spaced cracks.

The five crack spacing classes adopted are as follows :

I spacings between 0 and 0.60 m (0 and 1196 ft)
II spacings between 0.60 and 1 m (1.96 and 3.28 ft)
III spacings between 1 and 4 m (3.28 and 13.1 ft)
IV spacings between 4 and 10 m (13.1 and 32.8 ft)
V spacings in excess of 10 m (32.8 ft)

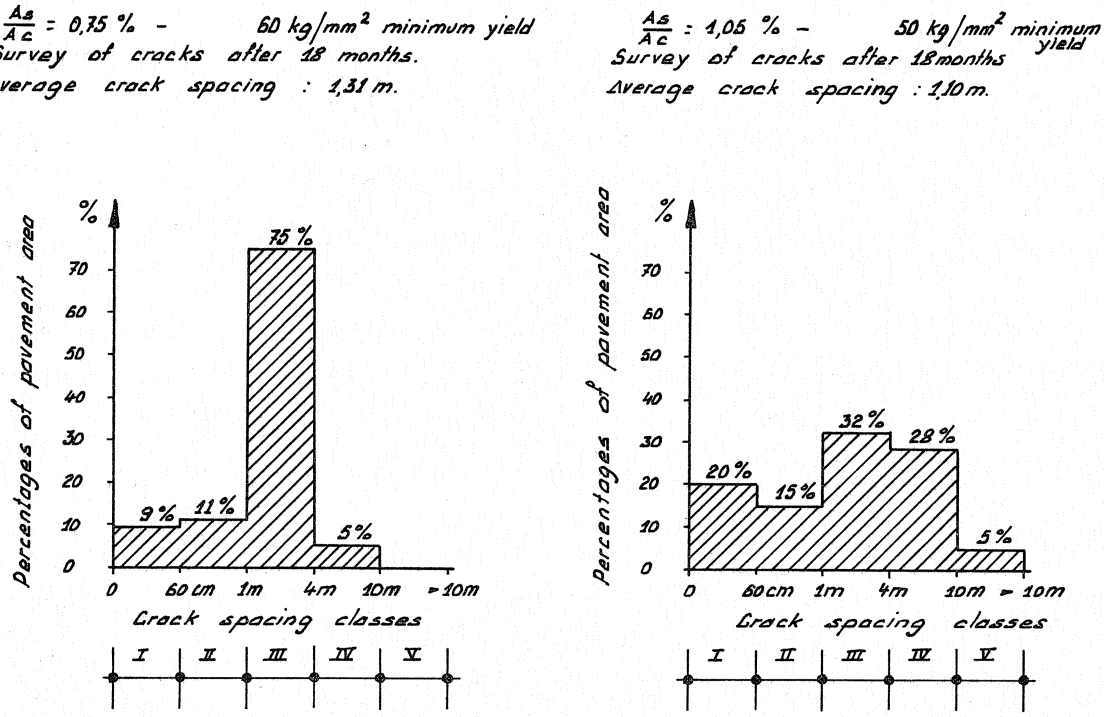
An ideal continuously reinforced concrete pavement should only contain cracks within spacing class III, while on the other hand classes I and V should be as small as possible, since the former of these two represents the condition where the cracks are too close together, with a risk that they will become bunched together and join up with one another, and the latter represents the condition where the cracks are clearly too far apart, such as may temporarily occur during the first few weeks after the pave-

ment has been put into service or, more permanently, when the longitudinal reinforcement is distinctly under-designed.

The survey that was carried out related to eight experimental sections of pavement which had been built between 1950 and 1971 and which had percentages of longitudinal reinforcement varying between 0.3 % and 1.05 %. Also comprised in the survey were six sections of motorways which had been chosen as being representative of the motorway pavements built in recent years and described in Chapter 2. The object of the survey was to collect the results of crack investigations that had been carried out on these road sections in course of time and to present them in the form of histograms indicating the distribution of the cracks over the five spacings classes defined above.

By way of example, Fig. 3 shows, for comparison, two very different histograms obtained after 18 months of service : the first histogram testifies to the excellent regularity of cracking on the section of road concerned, while the second reveals the heterogeneity of cracking on another section of road and also shows how high a percentage of the latter is affected by too closely spaced cracks.

Fig. 3



The main conclusions that can be drawn from this survey are the following:

- Cracking of continuously reinforced concrete pavements continues to develop during the first few months after construction and reaches something like a state of equilibrium at the end of 2 - 3 years.

- For reinforcement with a yield point of 50 kg/mm2 the average crack spacing (A.C.S.) on all the pavements examined is directly linked to the percentage (p_s) of longitudinal reinforcement by the formula :

$$p_s = \frac{1}{\sqrt{A.C.S.}} \qquad \text{(Fig. 4)}$$

It follows from this that the reinforcement percentage should be reduced from 0.85 % to 0.70 % in order to obtain optimum crack spacings

of between 1.50 and 2.50 m.

- As a result of using reinforcement with a higher yield point of 60 kg/mm2 instead of 50 kg/mm2 the percentage of longitudinal reinforcement may be reduced. The average crack spacings obtained in that case, however, are smaller than those calculated from the above formula.

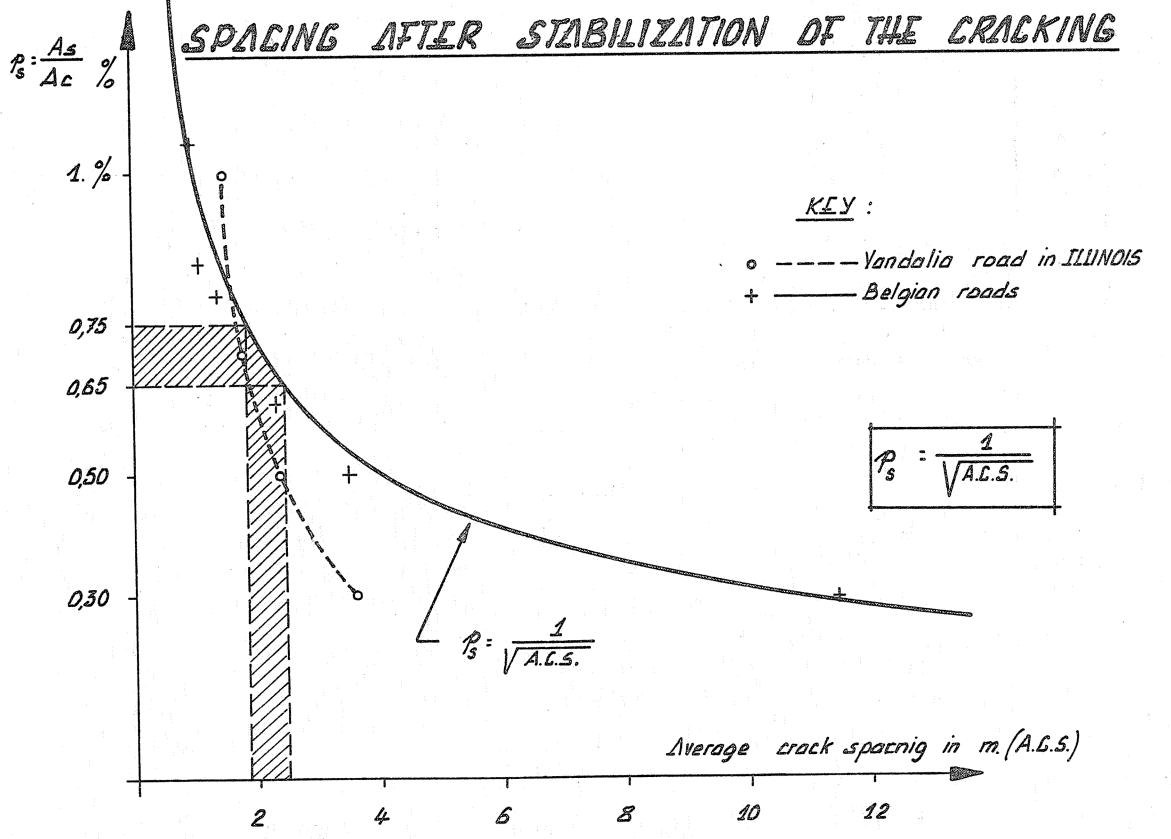
- As regards the distribution of the cracks, the pavements examined in the survey can be classified in three distinct categories :

  * Sections of road on which the longitudinal reinforcement is distinctly under-designed (p_s = 0,3 to 0,5 %) and on which the too widely spaced cracks (corresponding to classes IV and V) affect 50 - 100 % of the overall pavement area, whereas the parts with closely spaced cracks (classes I and II) comprise a very small and indeed negligible area.

  * Motorway sections which have been constructed in recent years and which generally present a rather heterogeneous crack distribution. Three years after construction, the very closely spaced cracks of class I affect 25 - 30 % of the area of these pavements, while no more than 50 % of the total area ever has cracks with optimum spacing corresponding to class III.

Fig. 4.

RELATION BETWEEN PERCENTAGE OF LONGITUDINAL REINFORCEMENT AND THE AVERAGE CRACK SPACING AFTER STABILIZATION OF THE CRACKING



* Some experimental sections with optimum crack spacing, 70 - 80 % of the area having cracks spaced at between 1 m and 4 m apart. On these sections the percentage of longitudinal reinforcement ranges from 0.70 % to 0.85 %.

- Lowering the level of the layer of reinforcement from the upper third of the slab thickness to mid-depth thereof appears to have a favourable effect on crack distribution.

- The predominance of crack spacing class I on motorway sections probably indicates the effect of traffic on the distribution of the cracks. The tensile stresses set up in the concrete by the traffic loads are superimposed on those due to the variations in the temperature and the moisture content of the concrete of the pavement.

3.2 Laboratory Investigation Of Steel-to-Concrete Bond Phenomena In The Vicinity Of Cracks In Continuously Reinforced Concrete Pavements.

The object of this research was to pin-point the various factors affecting the cracking of continuously reinforced concrete pavements and more particularly to explain the appreciable difference in behaviour between the American continuously reinforced pavements, where the mean distances separating cracks range from 1.5 to 3 m, and the ones built in Belgium, where the average crack spacings are much smaller. To this end, the investigation was directed at experimentally determining the anchorage length from Vetter's formulas, this being the length along which the stresses are transferred from the steel to the concrete on each side of a crack. The experimental procedure adopted was based on the conventional pull-out test, appropriately modified in a novel manner to achieve perfect simulation of the stress conditions to which the two materials are subjected in the pavement (Fig. 5).

By means of successive approximations a bond length of such magnitude is determined that the relative slip between the bar and the concrete behind the test specimen is about to occur when the pull exerted on the bar corresponds to the maximum stress (σ_s) attained in the bar at the instant of cracking of the pavement. This bond length is by definition the anchorage length if the design of the test specimen corresponds to that of a continuously reinforced pavement with :

$$p_s = \frac{As}{Ac} = \frac{\sigma_c}{\sigma_s - n.\sigma_c} \qquad (1)$$

p_s = As/Ac = percentage of longitudinal reinforcement ;

n = ratio of the moduli of elasticity of steel and concrete ;

σ_c = direct tensile strength of the concrete.

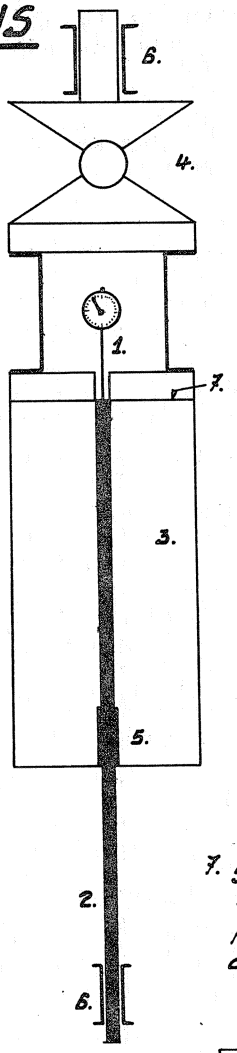
The variables in the tests were,on the one hand, the composition of the concrete - American and Belgian highway concrete (see § 2.4) - and, on the other, the age at testing (7, 14 and 56 days).

For the two types of concrete and for the ages at testing, the relation between the length of the specimen (l) and the tensile stress (σ_s) acting in the reinforcement at the precise instant when slip begins behind the specimen is presented in Figs 6 and 7 respectively. When the tensile strength of the concrete is known, it is easy to deduce the corresponding stress acting in the steel at the crack at the instant when the concrete reaches this failure limit, and then to interpolate or, if necessary, to extrapolate the experimental curve (σ_s - l) a little in order to find the corresponding anchorage length l_a.

Fig 5

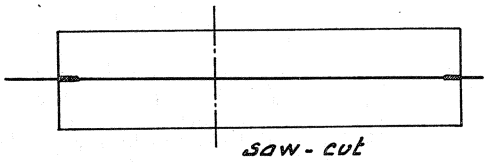
TESTING APPARATUS

1. measurement of relative slip between steel and concrete behind the specimen. in 10^{-3} mm
2. 16 mm diameter reinforcing bar $A_s = 2.01\ cm^2$
3. prismatic test specimen $A_c = 15.8\ cm \times 15.8\ cm = 250\ cm^2$
4. hinge
5. plastic sleeve to elimiante wall effect (5cm)
6. tension gripping jaws



$$\rho_s = \frac{A_s}{A_c} = \frac{2.01}{250} = \frac{0.804}{100}$$

7. glued face of specimen; this is the sawn face of the prism as indicated in the drawing below



MANUFACTURE

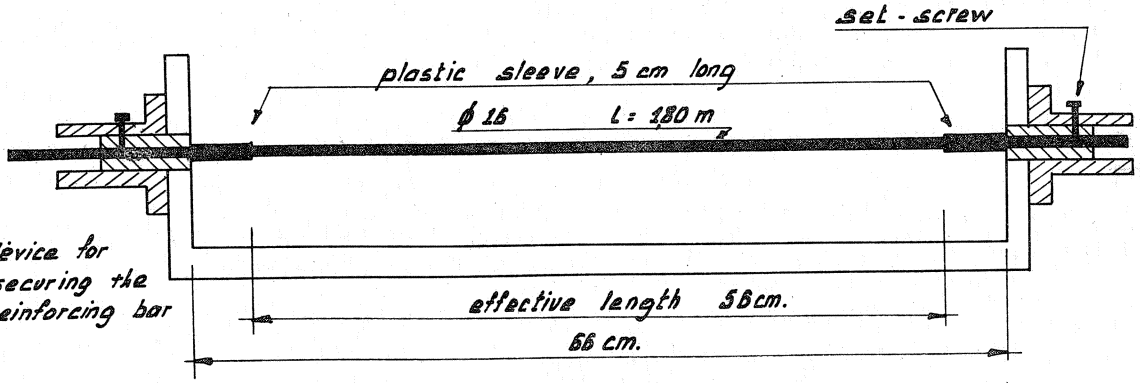


Fig. 6.

VERY HIGH COMPRESSION STRENGTH BELGIAN ROAD CONCRETE

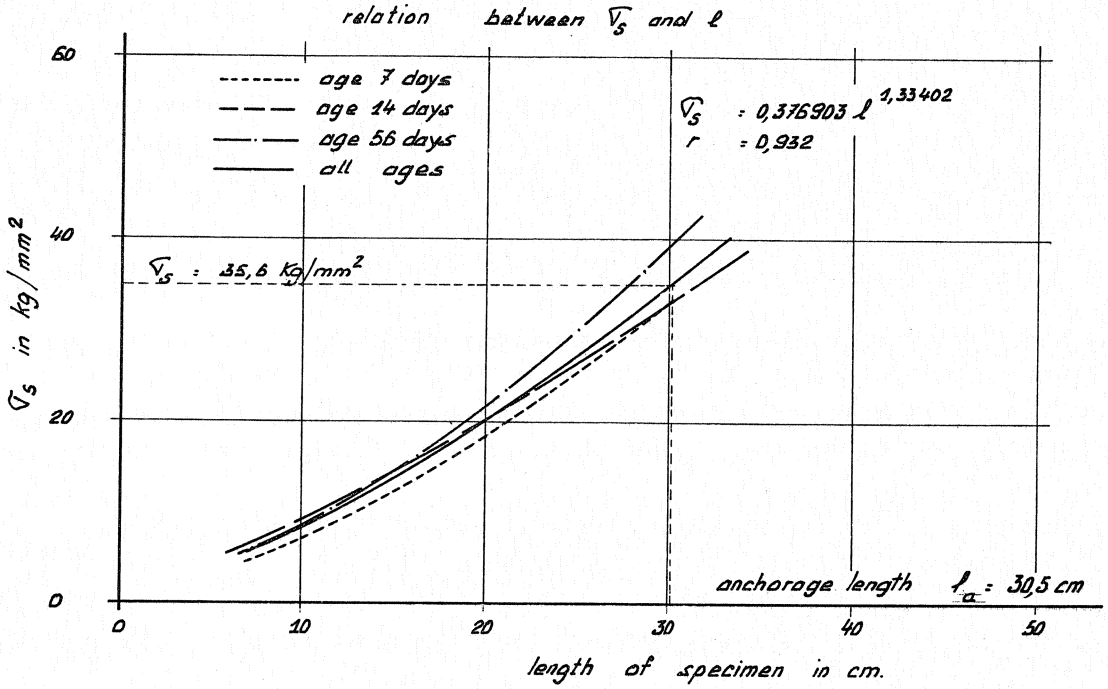
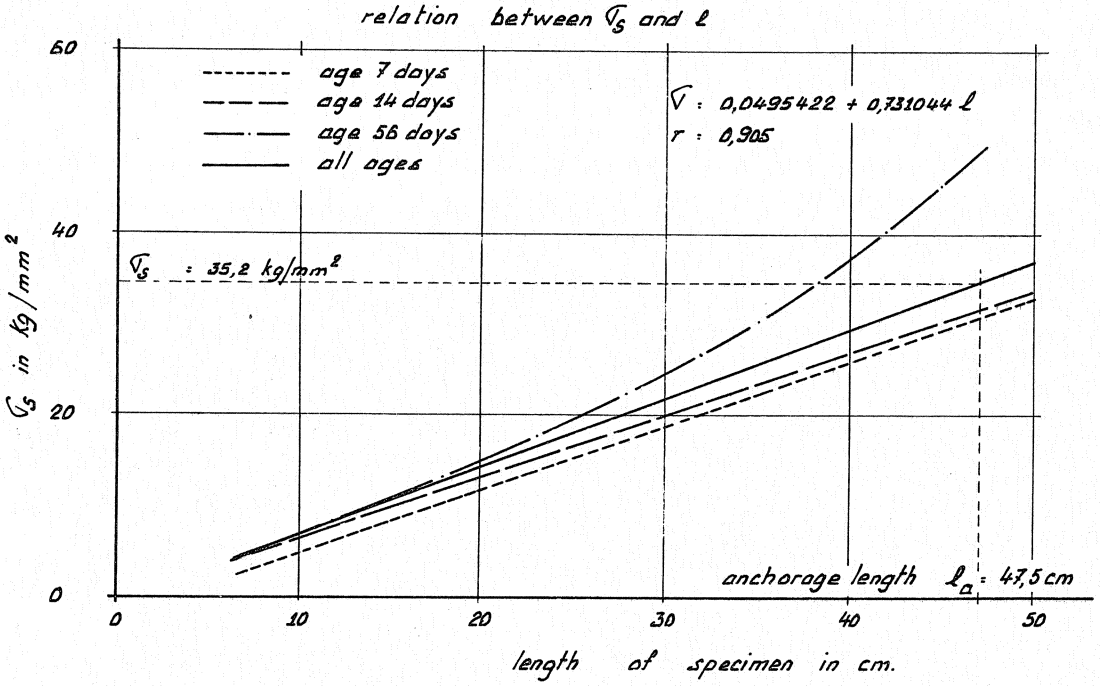


Fig. 7.

AIR-ENTRAINED AMERICAN ROAD CONCRETE



In reality the direct tensile strength of the two concretes tested is practically identical and independent of the age between 7 and 56 days : σ_c = 27 kg/cm2.

Adopting n = 6 for the Belgian (E_c = 350 000 kg/cm2) and n = 7.5 for the American highway concrete (E_c = 280 000 kg/cm2), we obtain from formula (1) and Figs. 6 and 7 :

- for the Belgian concrete : σ_s = 35.6 kg/mm2 and $l_a \cong$ 30.5 cm

- for the American concrete: σ_s = 35.2 kg/mm2 and $l_a \cong$ 47.5 cm

The age of the concrete between 7 and 56 days has no ascertainable effect on the anchorage length, just as it has practically no effect on the direct tensile strength of the concrete.

Besides, the anchorage length does not appear to be linked to the tensile strength of the concrete, since the anchorage lengths for the two types of concrete tested differ greatly, whereas the tensile strengths of these concretes are very nearly identical.

Nor is the anchorage length linked to the compressive strength of the concrete, because for each of the concretes investigated here the anchorage length is practically independent of age and because age has a major effect on their compressive strengths.

The notable difference between the anchorage lengths measured for the two concretes must therefore be essentially due to the difference in their mortar content and especially to the presence of entrained air in the American concrete.

However that may be, with the introduction of measured anchorage lengths into C.P. Vetter's established formulas it is possible to explain the different behaviour of the American and Belgian continuously reinforced concrete pavements. The calculation leads to the following conclusions :

- in Belgium with p_s = 0.85 % the average crack spacing is approximately 1 m ;

- in the United States with p_s = 0.70 % the average crack spacing is approximately 2.30 m.

The average crack spacing on the Belgian pavements is thus about 55 % less than the corresponding figure for the United States, half this difference being due to the difference between the concretes used in the two countries and half to the difference between the respective reinforcement percentages.

These figures calculated from anchorage length measurements correspond in any case fairly closely to reality as observed in both countries. The Vandalia road section in Illinois, with 0.70 % longitudinal reinforcement, has an average crack spacing of about 1.80 m after 20 years of service. On the other hand, only a few months after being put into service the pavements of motorways constructed in Belgium already have an average crack spacing of between 1 and 1.50 m.

3.3 Longitudinal Cracks.

Erratic longitudinal cracks have appeared on the majority of concrete pavements constructed across a large width - corresponding to two or three traffic lanes - in a single concreting pass. These cracks straddle the

longitudinal dummy-type warping joints always formed between the several traffic lanes but whose preformed initial depth of 3 to 5 cm at the top of the slab has turned out to be quite insufficient. Fortunately, the presence of the transverse reinforcing bars of the continuously reinforced concrete renders these longitudinal cracks fairly harmless because their width is strictly limited.

It has been established that these cracks are attributable to the superposition of the warping stresses due to the thermal gradient through the thickness of the slab and the stresses due to the traffic loads.

These cracks always start at the bottom fibre of the slab and can unite with the preformed upper initial part of the longitudinal joint only if this preformed depth is great enough - at least equal to a third of the slab thickness. It is preferable to preform the joint by sawing into the hardened concrete rather than by merely inserting a strip of polyethylene into the fresh concrete, as the effectiveness of this latter method of forming a break in continuity in the compressive upper zone of the concrete at the time of cracking is disputable.

3.4 Corrosion Of The Reinforcement In Continuously Reinforced Concrete Pavements.

In 1974 a core drilling campaign was launched on seven experimental road sections, ranging in age from 3 to 24 years, with a view to investigating the state of preservation of the reinforcement and to assessing the risks of corrosion under the effect of the increasingly intensive use of chemical de-icing agents in winter. The sampling operations comprised the taking of 71 samples of longitudinal and 40 of transverse reinforcing bars. For reasons of comparison these samples were taken at the transverse cracks themselves and in uncracked zones of the pavement.

The corrosion of the reinforcement was evaluated on the basis of visual inspection and of optical measurements. The main conclusions from this investigation are as follows : (ref. 5).

- The longitudinal bars which orthogonally intersect the characteristic cracks of continuously reinforced concrete are very little attacked by rust. On the other hand, the transverse bars are more vulnerable in so far as they initiate cracking of the concrete ; 24 % of the longitudinal bars examined, and 50 % of the transverse bars, show signs of corrosion. However, in the majority of these bars the corrosive attack is only very local and fairly slight : in no case is the steel cracked, and the pits observed in the steel are for the most part less than 0.5 mm in depth.

- The width of the cracks certainly constitutes the dominant parameter with regard to the durability of the reinforcement, inasmuch as corrosion remains slight so long as the crack width above the bar remains less than 0.5 mm. In so far as the percentage of longitudinal reinforcement is sufficient to prevent excessive crack widths the corrosion of the steel does not present a hazard to continuously reinforced concrete pavements.

- There is practically no trace of rust on reinforcing bars taken from uncracked zones of the pavements. This absence of rust is due to the fairly considerable depth of concrete cover (60 - 85 mm) and to the high compacity of the highway concrete concerned.

3.5 Evenness And Roughness.

On some motorway sections the pattern of the reinforcing bars seems to manifest itself at the surface of the concrete, thereby producing systematic waviness with an adverse effect on riding comfort which is addi-

tional to the effect of the transversely grooved surface finish. This phenomenon is especially noticeable on stretches of road with considerable longitudinal gradient (5 - 6 %) which were concreted by slipform pavers working in the uphill direction.

These irregularities are attributed to two distinct causes. First, there is the decompression of the concrete as it emerges from under the last mechanical compacting device of the paving machine ; this decompression causes the fresh concrete to swell or expand somewhat more between the reinforcing bars than over them. The second factor is that vibration is set up in the reinforcement which in turn causes "over-vibration" of the concrete over the reinforcement and thus produces the irregularities observed.

The evenness of continuously reinforced concrete pavements, as this of unreinforced concrete pavements, is sometimes also affected by more or less extended settlement in consequence of poor subgrade or of after-compaction of fill material. In most cases there is considerable local differential settlement, from 5 to 10 cm, over a distance of about ten metres at each end of bridges and similar structures that the pavement crosses without any break in its continuity. Sometimes, however, settlement effects of greater wavelength (5 - 15 cm over distances of 40 - 100 m)occur on high embankments or on subgrade of very low bearing capacity. In either case the excellent behaviour of the pavement in such zones of settlement is a notable feature, this being due to the low rigidity of the continuous slab in the longitudinal direction because it develops a large number of cracks which function as hinges.

Such subsidence effects must be eliminated, however, in order to restore the road surface to its correct levels and to obviate the development of eccentric compressive forces in hot weather which could cause blow-ups at points where considerable short-wave settlement had occurred. Slab jacking with cement grout (6 - 8 kg/cm^2 pressure) provides a particularly simple method to restore the original pavement profile and some rather spectacular examples of corrective treatment by this method, without any adverse consequences to the pavement, are on record.

3.6 Local Deterioration Of Concrete Due To Defects Of Execution.

Exceptionally it occurs that disintegration of the concrete above the reinforcement, affecting an area not exceeding a few square metres, makes it necessary locally to replace the concrete throughout the full thickness of the slab. Such trouble is due to defects of execution and is more particularly attributable to the presence of concrete that was locally not vibrated and compacted sufficiently - for example, at construction joints.

A quick method of patching has been developed based largely on the American recommendations published by the Continuously Reinforced Pavement Group. However, in order to reduce as much as possible the interruption of traffic due to these repairs, a special concrete mix has been designed which possesses good workability at the time of placing but moreover ensures that the necessary strength to allow traffic to travel over the repaired parts is attained in a short time (between 24 and 48 hours). The minimum required strength for this purpose is estimated at 400 kg/cm^2, measured as compressive strength on cylinders 10 cm in height and 100 cm^2 in cross-sectional area. Besides, this concrete mix is based exclusively on the use of materials readily available at ready-mixed concrete plants.

4. CONCLUSIONS

Motorway construction in Belgium has greatly slowed down since 1974. Fig. 1 shows, however, that substantial lengths of motorway to be built in continuously reinforced concrete are planned for the years ahead. Besides, the Ministry of Public Works is now intent upon modernizing the State's primary highway network, and continuously reinforced concrete can once again provide excellent solutions for that modernization in the form of overlays to bituminous or to concrete pavements, with or without widening of the carriageways. In this respect, too, Belgium is following in the steps of the United States where this technique of overlaying existing pavements with continuously reinforced concrete is already highly developed.

In our country a number of such road sections have already been in existence for 5 to 10 years, and their behaviour is entirely satisfactory. Some versatile and very handy slipform paving machines specially designed for the construction of 3 to 8 m (9.8 to 26.2 ft) wide slabs have recently appeared on the Belgian market in anticipation of such roadmaking work.

Following the experience gained from analysing the behaviour of existing pavements, the following principal modifications will be applied to the design of new road sections to be built in continuously reinforced concrete :

- Reduction of the longitudinal reinforcement from 0.85 % to 0.67 % for steel with a yield point of 50 kg/mm^2 (71,100 psi).

- Lowering the level of the reinforcement to mid-depth in the slab in order to obtain better distribution of transverse cracks and better riding comfort.

- Increasing the preformed depth of longitudinal warping joints (between traffic lanes) to at least one-third of the overall thickness of the concrete.

In addition, research on the following subjects is to be carried out :

- The possibility of replacing the transverse grooving by shallower brushed finish, or by the recently developed treatments of the fresh concrete by chipping or aggregate exposure, with a view to improving the riding comfort of the pavements and lowering the noise level.

- The possibility of reducing the reinforcement percentage to around 0.50 % by using steel with a higher yield point : 60 kg/mm^2 (85,400 psi) as against 50 kg/mm^2 (71,100 psi).

5. ACKNOWLEDGMENTS

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Having since applied this technique on a large scale in Belgium for a number of years, the present authors are very grateful for the opportunity afforded them by this International Conference at Purdue University to report in their turn upon the results of their own experience and thus to continue a fruitful dialogue.

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