ON THE USE OF PRESTRESSED CONCRETE IN EARTHQUAKE RESISTANT DESIGN

by

J. DESPEYROUX (°)

ABSTRACT

The aim of the present paper is to set, without any thought of exhausting the subject, the problem of the resistance of prestressed concrete structures to earthquake motion.

It draws up the list of the main objections sometimes counted against this material, and of the reasons which are in favour of the possibility of using it in aseismic design.

FOREWORD

by

Y. GUYON
President of the F.I.P.

The paper by Mr. DESPEYROUX raises the main objections which have been opposed to the suitability of prestressed concrete to antiseismic construction.

After having read this paper as well as various reports on the behaviour of different prestressed structures in the recent earthquake, I come to the conclusion that the main problem to be solved for the development of application of prestressed concrete to antiseismic structures is not the technical one - although it exists - but a problem of education.

The fact that Mr. DESPEYROUX as well as Professor T. Y. LIN (in a paper read at a ASCE-PCI joint conference) are obliged to demonstrate the ability of prestressed concrete to resist statically inversion of loads, is simply astonishing for me - it is so obvious, and a pure

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question of position and force of the cable : in prestressed concrete we have always two extreme states of stresses, and they are precisely opposite. Under seismic conditions they will be, if I can say, a little more opposite (which means that the interval between the negative and the positive will be slightly greater) and that is all.

On another hand, the fact that very heavy mistakes from the seismic point of view have been made in the structures which have collapsed in Anchorage shows a lack of knowledge, on the part of their designers, of the fundamental requirements of seismic resistance. It is not the material which is faulty, but the engineering which is seismically bad.

The perfect behaviour of 15 prestressed structures in Anchorage and of 3 structures which Mr. SUTHERLAND has scrutinized in the last Tokyo earthquake demonstrates the suitability of the material when correctly used. In the cases where it behaved badly, the engineering fault is obvious : absence of transversal resistance of the walls, bad anchorage of bars, and also faulty welded assembling, for which, it is certain, the designer has not thought enough on the stresses to which they could be submitted. But this is not prestressed concrete, and failure would have occurred with any other material - and in fact, has actually happened when mistakes of the same kind have been done.

Therefore, this liaison between seismic and prestressed concrete engineers is essential and it is for this reason that F.I.P. has set up its Committee for seismic resistance. I feel sure that, under the chairmanship of the eminent seismic specialist Professor MUTO, and the vice-chairmanship of Messrs. DESPEYROUX - another very proficient seismic engineer - and SUTHERLAND - a very experienced engineer in prestressed concrete, excellent work will be achieved.

Mr. DESPEYROUX's paper is in itself a list of the basic ideas on which a mutual agreement must be reached, not only by the reasoning, but by tests.

It is certain, for instance, that, on the matter of the flexibility, the reasoning according to which this would be rather an advantage, may be checked by models, and perhaps, to avoid disorders in the non-structural elements, the use of flexible connections could be considered.

For the energy dissipation, from what I have read, the unanimity is far from being made. Some say that dissipation is of the same order for prestressed concrete and for reinforced concrete (DESPEYROUX, LIN), some say no.

It seems that there is a confusion. Of course, as long as the prestressed concrete has not cracked, the energy dissipation is smaller, and much smaller than for the reinforced concrete, but as long as it has not cracked, its behaviour is perfect. Therefore, the comparison must be made after cracking, and up to the limit state of pre-rupture. In this line, it is a question of the moment-curvature diagram and more precisely of the area under this diagram, which is a measure of the energy dissipated. This area may be greater for R.C. than P.S.C., or inversally ; it is a question of percentage of steel. With high percentages it will be smaller.

It is also a question of the kind of prestressed concrete (partial or total).

But again, tests are needed and any a priori idea, for or against, must be confronted with experience.

We must of course be ready to accept, with an entire intellectual honesty, the consequences of this confrontation.

I feel sure that an experimental work along these lines, carried out and interpreted in a full cooperation between seismologists and prestressing engineers, will be extremely beneficial. It is urgent to undertake it, since it appears that seismologists come to prestressed concrete or at least begin to think more and more of it, on account of its advantages as far as recovery is concerned.

I hope that Mr. DESPEYROUX's paper will convince those who would have to be convinced.

ON THE USE OF PRESTRESSED CONCRETE IN EARTHQUAKE RESISTANT DESIGN

by J. DESPEYROUX

Our experience of the behaviour of prestressed concrete structures during an earthquake is not very large. Before the Alaskan earthquake of March 1964, we never had an opportunity of observing prestressed elements submitted to earthquake motion. Therefore, the engineers have quite different opinions about the suitability of prestressed concrete to earthquake resistant design.

Most of the antiseismic construction codes existing in the world do not make any explicit mention of prestressed concrete. From the silence of the rules, some people infer that this material is not particularly appropriate to antiseismic construction. Some of the arguments on which they found their feeling are listed below.

On the other hand, some codes like the French code entiled "Règles Parasismiques P.S. 64" explicitly admit prestressed concrete, without providing any special limitation about it.

The aim of the present paper is to set the problem of the utilization of prestressed concrete in aseismic design, in order to raise discussions and researchs about this very important subject.

The main objections counted against prestressed concrete in aseismic design are the following :

1°) - Prestressed concrete is not fitted for satisfactorily resisting the reversed loadings due earthquake motion ;

2°) - Prestressed concrete members are too flexible compared to members of the same bearing capacity made with other materials. This flexibility may occasion damage in brittle non-structural elements.

3°) - The ductility of the material is not large enough and its energy absorbing capacity is rather low.

4°) - It is difficult to arrange satisfactory joints between prestressed members, especially in corners of frames.

The French code of 1964 having been worked out before the Alaskan earthquake of March 1964, the reasons upon which its authors founded themselves when making provisions for prestressed concrete structures, are theoretical ones. It is very likely that their conclusions would have been the same if they had known the effects of the earthquake on the buildings of Anchorage in which prestressed concrete had been used.

RESISTANCE TO REVERSED LOADINGS

In earthquake resistant design, the problem of the resistance to reversed loadings is set more especially for columns and short parts of girders around them, as a consequence of the alternate lateral forces acting upon the structure.

In its principle, this problem is not different for the various materials used in construction technics. It is to be set exactly in the same way for reinforced concrete members, or even for steel construction (in the case of dissymmetrical cross-sections submitted to alternate bending moments) as for prestressed concrete.

It has to be put not only for earthquake motion, but also for lateral wind forces. Prestressed structures being able to resist wind forces, they must also be able to resist earthquake lateral loadings. As long as the resulting stresses are below the yield point of the various elements of the structure , the loading is similar to a fatigue loading. Earthquakes being by chance unusual events, and being, each time, of rather short duration, the number of reversals due to earthquake motion during the life of a structure is small compared to reversals due to other causes, and insignificant compared to the possibilities of the material.

From this point of view, the main difference with the resistance to wind effects is that, in the event of an earthquake of intensity larger than the expected intensity, some plastic yieldings may occur. The point is then to decide whether these yieldings alter in a significant manner the resistance of the elements to stresses acting in the opposite direction.

Tests have been made upon reinforced concrete elements. They showed (1) (2) that the resistance to reversed loadings remains unchanged in so far as the element has not been laden beyond 80 % of its ultimate strength. There is no reason why these conclusions could not be applicable to prestressed concrete, and it is to be desired that tests would be done in order to clarify this point.

The real problem of the resistance to reversed loadings is a problem of adequate code provisions, in order that every cause of reversed loading could be correctly taken into account. It is precisely in order to forecast as exactly as possible all the reversals of loadings that the French code attaches such a large importance to the vertical components of seismic forces : in this code, vertical seismic forces are supposed to act concurrently with the horizontal ones.

'LEXIBILITY OF PRESTRESSED CONCRETE MEMBERS

Prestressed members are generally more flexible than reinforced concrete ones of the same bearing capacity. It is necessary to note - though it is obvious - that flexibility is not a defect in itself since t is always possible to modify the cross-sections characteristics of a structure, or even its type, in order to reduce its deformations.

According to modern principles of earthquake engineering, the response of a structure to earthquake motion depends upon the periods of its natural modes of vibration. The longer these periods, the smaller the seismic forces acting upon the structure.

From this point of view, flexible prestressed structures, the periods of which may be long, appear as less sensitive to seismic action, than stiff ones. Moreover, the dead load being reduced, the seismic forces may be noticeably smaller in the case of prestressed structures.

Of course, excessive deformability may be a cause of damage in non-structural elements, but this time again, the problem is the same for the various structural materials. It behoves the code to lay down proper drift limitations.

Most of the modern codes made such provisions for the relative displacement between two consecutive floors. They also include provisions concerning the width of the separation to be saved between adjoining buildings, in order to prevent hammering.

In some cases, the too large flexibility of a structure may occasion supplementary stresses, especially in columns, where supplementary moments - sometimes called "second order moments" - may appear. This is not peculiar to prestressed concrete.

DUCTIBILITY AND ENERGY DISSIPATION

Many people consider that prestressed concrete has a small energy absorbing capacity. As there are only a very few experimental data about this point, this statement seems to be a matter of feeling rather than an opinion supported by experience.

In fact, in a prestressed concrete member, as long as cracking does not occur, energy can be dissipated only in concrete, since before cracking the strain of prestressing steel remains so to speak unchanged. The contribution of steel to energy dissipation begins only after cracking. So, it may be feared, at first sight, that prestressed concrete would be a less valuable material that some others.

However, when comparing prestressed concrete to other materials, it is necessary to compare things worth to be compared, that is members having the same bearing capacity. Such a comparison has been done, as so far as it is possible to make such a comparison, in fig. 1. The curve "a" is an actual experimental curve obtained in the University of California by CAULFIELD and PATTON (3). The curve "b" is a theoritical one obtained from computations by the writer for a reinforced concrete beam having a yielding moment equal to the cracking moment of the first.

Since the energy stored per unit of length can be written as the product of moment M by curvature ω the areas under the curves give estimations of respective energy absorption capacities. There is no reason, until it has been decided by experience, to say that the area under curve "b" (prestressed member) is systematically smaller than the area under curve "a" (reinforced concrete member). In many cases, it may be the contrary.

It may be seen also that the ductilities defined as the ratio of curvature at ultimate to that at yield for reinforced concrete, or at 0.6 ultimate for prestressed concrete, are of the same order.

It is also necessary to point out that if we consider the influence of damping properties over the response to earthquake motion, the parameter which is significant is not the damping coefficient in itself, but the damping ration, i.e. the ratio ν of the damping coefficient to the critical damping. Prestressed members being both lighter and more flexible than reinforced concrete ones, their critical damping is smaller, so that the damping ratio in prestressed concrete is larger, that is to say more favourable, than it could be thought from the mere consideration of damping coefficients.

Moreover, it is to be noticed that, in the case of buildings, energy dissipation takes place not only in structural members, but also, to a certain extent, in non-structural elements, chiefly in the form of external frictions. It is a well known fact that the damping factor ν , the value of which is 1 to 3 % of critical in bare steel structures or 8 % in bare reinforced concrete structures, may rise respectively up to 5 and 15 % in buildings (4).

For the various reasons exposed above, it seems that, all things considered, the difference between prestressed and reinforced concrete is not so large as it has sometimes been said.

JOINTS

In prestressed structures, the design of joints requires careful consideration, especially in corners of frames. However, these problems are now well known. Some tests have clarified the behaviour of such joints, and typical examples may be found in litterature (see by instance ref. 5). In this matter, cleverness in engineering is of primary importance.

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For the various reasons exposed above, it seems that prestressed concrete is as suitable to earthquake resisting design as some other materials. Of course, its properties have to be checked by tests.

In Anchorage, there were twenty seven buildings in which prestressed concrete was used. But it was used only in floors generally supported by a concrete skeleton or masonry walls. Thus, though twenty one of these buildings went trough the earthquake with very few damage or no damage at all, we cannot say that the Alaskan earthquake has brought a large contribution to our experience of the behaviour of prestressed structures submitted to seismic motion.

However it is interesting to note that, among the six buildings which suffered heavy damage or collapsed, four failed for lack of an adequate lateral forces resisting system.

The other two were buildings under construction. One of them, the "Alaska sales and service" was made of reinforced concrete frames composed of precast elements assuming the shape of a T. These frames supported precast prestressed beams. The structure failed because the reinforced concrete beams which would have tied up the T frames and completed the bracing system, were not cast at the moment of the earthquake.

The resistance of prestressed concrete is not implicated in this failure.

The last one, called "the four seasons building" was a six-story lift-slab structure. The slabs were post stressed by means of unbounded tendons. The lateral stability depended upon two cast-in-place stair and elevator towers. Floors rested on steel columns by means of collars (Fig. 2). From the examination of the ruins, it is obvious that in a first step the floors slid down along the towers and the towers themselves tilted over a few moments after.

Some experts explain the collapse as a consequence of the rupture of one or several tendons, as a consequence of an hypothetic relative displacement of the towers when oscillating. This is not the opinion of the writer after a visit to Anchorage. On the assumption of such a displacement, it would be more convenient to set forth a loss of prestress due to the forces exerted by the towers when tending to move apart from one another ; but some computations show that this assumption is not likely. It seems to the writer that the explanation is more common and that the collapse occured because the joints between the slabs and the steel collars around the columns failed. In fact, for lack of an appropriate reinforcement, they could not withstand alternate curvature and subsequent shear stresses due to earthquake motion. The fall of the towers themselves may be a consequence of hammering during the collapse of the slabs, as well as of unadequate antiseismic design.

Besides Anchorage, it must be pointed out that there was a precast frame structure, with prestressed beams, in Cordova (at a larger distance from the epicentre). It behaved fairly well and was one of the few structures wich resisted the shock in Cordova.

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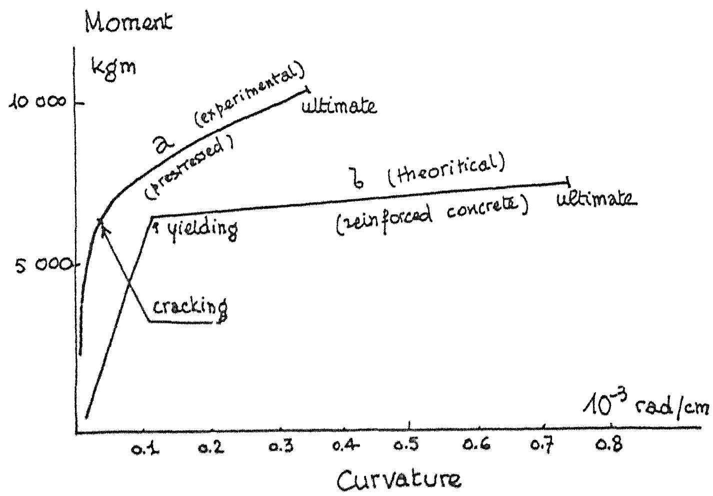

Moment
kgm
10 000
5 000
a (experimental)
(prestressed)
ultimate
b (theoritical)
(reinforced concrete)
ultimate
yielding
cracking
10^-3 rad/cm
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
Curvature

Fig 1

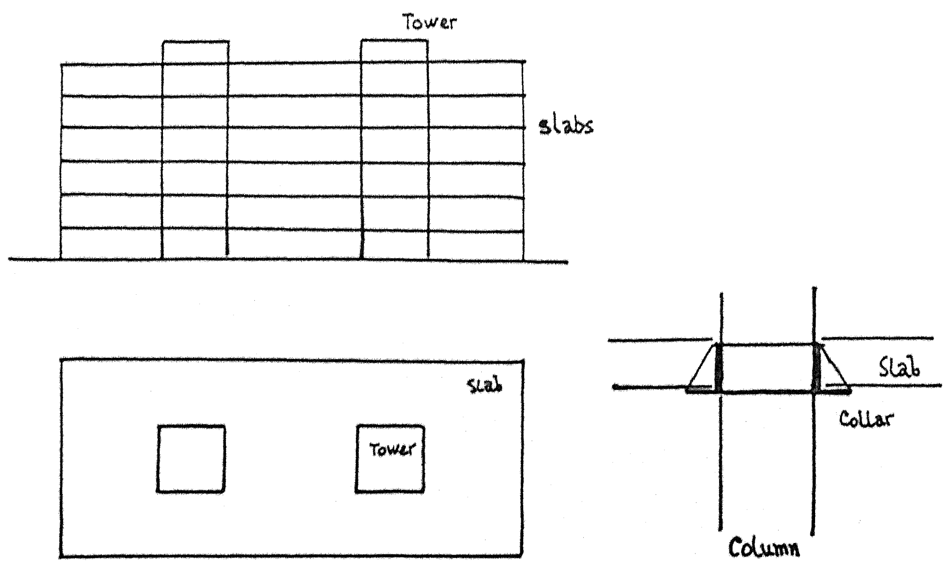

Tower
slabs
Slab
Tower
Slab
Collar
Column

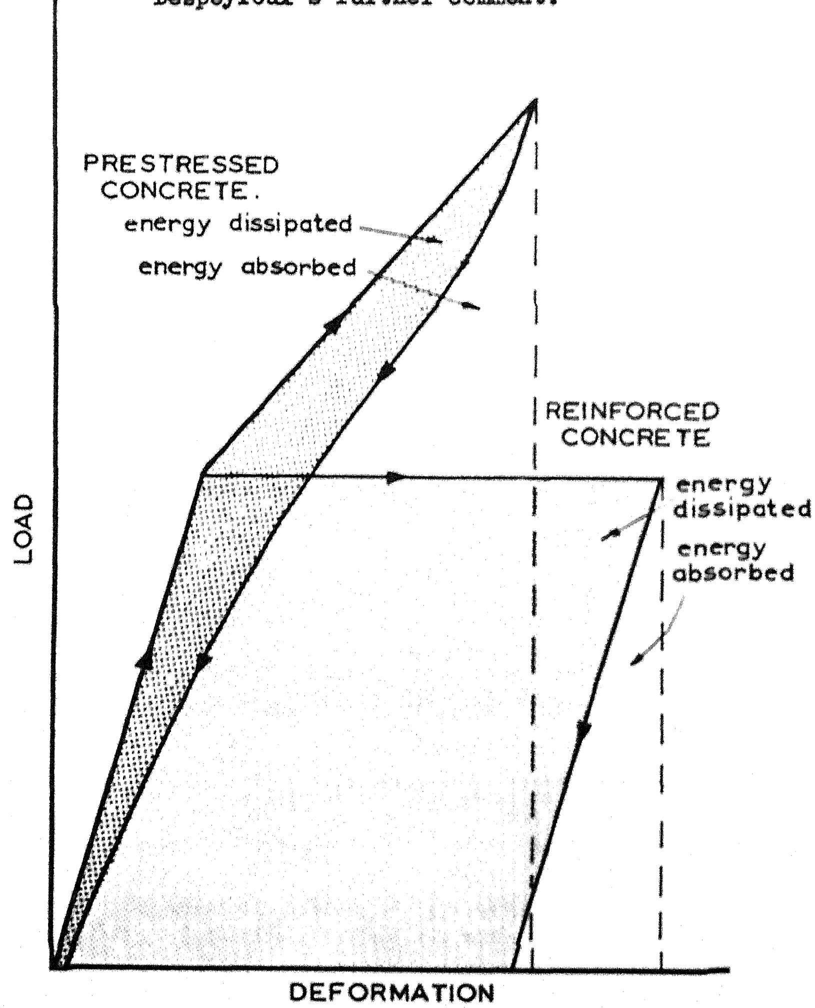
Fig 2

THE USE OF PRESTRESSED CONCRETE IN EARTHQUAKE RESISTANT DESIGN

BY J. DESPEYROUX

QUESTION BY: C.F. CANDY - NEW ZEALAND.

Repetitive loading tests on beams give load-deflection (Refers p. 6 and fig. 1) curves approximately as drawn below (for one cycle of load). The shaded area represents the energy dissipated; the area up to the dotted line represents the energy absorbed. Thus though the energy absorbed by a p-s member and a r-c member may be the same, very much more energy is dissipated in the latter member and thus the response of the latter to an earthquake must be smaller. It seems then that the reducing effect caused by plastic strain is much smaller in prestressed concrete. I would appreciate M. Despeyroux's further comment.



DISSIPATION OF ENERGY IN P-S. & R.C. MEMBERS.

AUTHOR'S REPLY: The problem is to know if, beyond the cracking point, there is a zone of the moment curvature diagram worth using from the point of view of energy dissipation. The first point is then to decide which is the limit state to be considered in the case of reinforced concrete and in the case of prestressed concrete. As, until now, there has been no general agreement about this point, it is only to avoid additional discussion that we compared the cracking and the yield points of each beam. In fact, there is no doubt that in a reinforced concrete member, it is hardly possible from the point of view of safety, to go far beyond the yield point, since, beyond that point there are probabilities of irreversible damage. In a prestressed member, it is possible to go very far, near ultimate, without altering the properties of the member in a significant manner for the future.

It is a fact that in the "elastic range" and perhaps in a certain zone beyond the cracking point, the energy dissipation in a prestressed member is small; but it is necessary to know what happens if we consider a larger interval. If the answer to this question is unfavourable, i.e. if the energy dissipation is small whatever the part of the diagram considered, then the problem of the utilization of prestressed concrete must be solved by taking into account seismic coefficients corresponding to a small degree of damping in structural elements. Damping in non-structural materials can favourably modify these coefficients.

But there is a much more interesting solution consisting of the addition of mild steel reinforcement.

Relatively high degrees of damping can be reached in this way.

QUESTION BY: B.H. FALCONER - NEW ZEALAND

Page 2, paragraph 3. Presumably the remark about the 3 structures relates to the June, 1964 earthquake. For that earthquake the discussor strongly doubts, from his own observations, that any inferences, either favourable or unfavourable, on the general suitability of prestressed concrete in earthquakes can be drawn since observed structural damage to whatever material was basically a result of foundation failure.

AUTHOR'S REPLY: I agree with Dr. Falconer on that point. The subject will be developed in the paper of Mr. Sutherland.

COMMENT BY: J.L. STRATTA - U.S.A. and J.A. SBAROUNIS

Both Messrs. Sbarounis and Stratta supported the theory that the lift shafts in the "Four Seasons Building" failed before the floor slabs fell to the ground; Mr. Sbarounis suggesting that it was due to a splice failure in the cores at the 1st floor level while Mr. Stratta maintained that one of the lift shafts had a far greater base fixity than the other, and thus this tower failed because it eventually took all the lateral shear force.

COMMENT BY: L. BENUSKA - U.S.A.

I take serious exception to your statement regarding the mode of failure for the "Four Seasons Building". It is highly unlikely that the floors slid down previous to the towers tilting over. There are three important considerations; the elevated mass, structural stiffness and lateral yield capacity of the system.

First, if the floor in fact had fallen away before the lateral failure of the towers, the reduction in elevated mass (about 90%) would have so greatly reduced lateral loading of the towers that they would have remained in their vertical position. Hammering during collapse of the floors, which you tentatively suggest caused the tower failures would be an insignificant lateral load on these elements with a total overturning yield capacity of about 43,000 kip feet.

Secondly, the towers were very much stiffer than the vertical load carrying frame composed of flat slabs and steel columns. The overturning moment at the base of the towers accompanying a roof sidesway of about 0.4 inches caused reinforcing steel in the tension flange of the towers to reach their yield stress. This magnitude of sidesway was of little consequence to the flexible vertical load bearing frame.

Thirdly, the yield capacity of the towers undoubtedly was not sufficient to resist the intense earthquake in an elastic manner, and ductile yielding of the towers was necessary to absorb the earthquake energy. The actual collapse of the building resulted from the overturning failure of the shafts near the base. All bars were spliced with a 20-bar diameter lap at one level. This investigator noted that the large bars, No. 8s and No. 11s, failed at the splice, while the No. 4 bars were broken with a ductile rupture. The 20-bar diameter splice would have resulted in an average bond stress of 560 psi when the steel was stressed to the yield point, a bond value which exceeds the ultimate strength values given in the A.C.I. code (318-63) for large bars. Bond failure with all bars spliced at one joint was brittle and could not provide the ductility necessary to absorb the energy imparted to the building by the serious earthquake.

AUTHOR'S REPLY: TO COMMENTS OF MESSRS. STRATTA, SBAROUNIS AND BENUSKA.

I think we must thank all the persons who took part in this discussion about the failure of the "Four Seasons Building" in Anchorage. As Mr. Guyon says in the fore-word of my paper there was a lot of wrong information and misinterpretation about the behaviour of structures where prestressed concrete was used. As various papers stated that in the "Four Seasons Building" the floors failed before the towers, it was interesting to imagine how such a failure could happen, and from my paper, you can notice how it was difficult to find a satisfactory explanation in this case. So, I am very happy to know that further investigations proved that the towers failed before the floors. This explanation agrees with my own feelings, that is to say that prestressing has nothing to do with the collapse of the building. By clarifying this particular point the contributors to the discussion have enabled the aim of this paper to be realised.