C A N A D A

PROVINCE DE QUÉBEC

COMMISSION D'ENQUÊTE SUR LE VIADUC DE LA CONCORDE

sous la présidence de : Me PIERRE MARC JOHNSON, Président
de : **M. ARMAND COUTURE, ing.,** Commissaire
et de : **M. ROGER NICOLET, ing.,** Commissaire

POUR LA COMMISSION :
Me MICHEL DÉCARY, procureur-chef
Me MARIE COSSETTE, procureur adjoint
Me JEAN-PATRICE DOZOIS,
Me POSEIDON RETSINAS.

Me MONIQUE MICHAUD,
greffière à l'audience.

LES PARTIES PARTICIPANTES :
Me PATRICK HENRY et **Me LAURENCE GAUTHIER**,
pour René Therrien, Gilles Dupaul, les employés
et les associés de Desjardins Sauriol & Associés.

Me PIERRE ARGUIN,
pour le ministre des Transports.

Me JEAN-CLAUDE HÉBERT et **Me DENIS DOLBEC**,
pour Inter State Paving inc.

Me ANDRÉ GUÉRIN,
pour la Ville de Laval.

LES PARTIES INTERVENANTES :
Me JEAN MORIN,
pour l'Association professionnelle des ingénieurs du gouvernement du Québec.

Me NORMAND D'AMOUR et **Me MATHIEU TURCOTTE**,
pour l'Ordre des ingénieurs du Québec.

M. DENIS DE BELLEVAL,
pour la Coalition pour l'entretien et
la réfection du réseau routier du Québec.

FL070709 **Le 9 juillet 2007.**

TABLE DES MATIÈRES

LISTE DES PIÈCES

2007-07-09

L'an deux mille sept (2007), ce neuvième (9e) jour du mois de juillet,

Me MONIQUE MICHAUD:

Vous pouvez vous asseoir.

Me PIERRE MARC JOHNSON:

Alors, maître Michaud, faites l'appel des procureurs. Merci.

Me MONIQUE MICHAUD:

J'invite les procureurs à bien vouloir s'identifier, en commençant avec le procureur-chef de la Commission.

Me MICHEL DÉCARY:

Messieurs les commissaires, Michel Décary et maître Marie Cossette.

Me PIERRE ARGUIN:

Alors, bonjour, Messieurs les commissaires, Pierre Arguin pour le ministre des Transports. Je suis accompagné de monsieur Jacques Gagnon, de l'ingénieur Bruno Massicotte et de maître Daniel Morin.

Me PATRICK HENRY:

Messieurs les commissaires, bonjour. Patrick Henry pour les ingénieurs concepteurs du viaduc et autres employés de Desjardins & Sauriol. Je suis accompagné de mon expert, le docteur Frédéric Légeron et de monsieur Gilles Dupaul.

Me ANDRÉ GUÉRIN:

Alors, bonjour, André Guérin. Je représente la Ville de Laval.

Me LUC CHOUINARD:

Luc Chouinard pour l'Ordre des ingénieurs du Québec.

M. GASTON PLANTE:

Gaston Plante en l'absence de maître Jean Morin pour l'Association professionnelle des ingénieurs du gouvernement du Québec. Je ne suis pas procureur.

Me PIERRE MARC JOHNSON:

Merci. Alors, Maître Décary.

Me MICHEL DÉCARY:

Alors, nous allons poursuivre avec le témoignage du docteur Mitchell. Auparavant, nous avons communiqué aux différentes parties et à leur procureur, une pièce, notre pièce COM-52-C, à laquelle il sera fait référence durant le témoignage du docteur Mitchell et ce document contient une copie de Info Structure, qui est une publication du ministère des Transports du Québec, et c'est celle du mois d'avril deux mille sept (2007). Donc, pièce COM-52-C. (**Pièce COM-52-C**) Et nous y reviendrons plus tard dans le témoignage du docteur Mitchell.

Me PIERRE MARC JOHNSON:

Est-ce que madame Desprez en a été avisée? Oui? Parce que je sais qu'elle a passé une partie de la fin de semaine à planifier nos travaux en termes de projection. Très bien.

In the year two thousand and seven (2007), on this ninth (9th) day of July:

TESTIMONY OF Mr. DENIS MITCHELL

WHOM after having made a solemn declaration, depose and say as follows:

EXAMINED BY Me MICHEL DÉCARY:

Q- Dr. Mitchell, when we left on Friday, we had concluded a section which addressed in particular the issue of the conception, the design of the structure, of de la Concorde structure.

Just as a starting point, could you remind us what the consensus was concerning the design of the structure?

A- Yes, certainly. On page 71, for those that have the books, there was consensus item 1.2.1 and this read:

> "*And agreed upon by all parties*
> *that in the design none of the*
> *major requirements were exceeded*
> *in satisfying the Code S6 1966.*"

In addition, the... no stirrups were required in the thick slab of the cantilever, that's 1.2.2, and the third one along those lines is that the norm, the Code in nineteen sixty-six (1966), did not contain any regulations for the design of disturbed regions.

Q- And if we come back a moment to 1.2.2, we will be coming back to this aspect, the fact that the norms in nineteen sixty-six (1966), nineteen seventy (1970), did not require that there be stirrups, *qu'il y ait des étriers dans le porte-à-faux*, in the cantilever.

So, we're now turning to page 74, which is... you're starting now a 3-D analysis with finite elements. Could you describe exactly what we'll be dealing with during the next remarks?

A- Yes, certainly. This was a more in-depth analysis of the bridge using more sophisticated tools. In this case, using finite elements, which I mentioned on Friday, is breaking the structure down into small little components, leaking them together and, by this way, one can predict what the distribution of forces are in the structure.

And what it does is it takes a count of the three dimensional geometry of the bridge, which is very important, particularly in this case, and it accounts for the stiffness of the different materials, in this case they're all concrete, except there are two different types of concrete in this case for the main structural components, and it takes account of the stiffness of all of the structural elements.

And, in addition to that, we can simulate with fairly reasonable assuredness the live load coming from the trucks and we can position the trucks in different locations on the bridge to find out the most significant loading cases.

I should point out that this is a linear elastic model. It doesn't account for any cracking. When we look at more sophisticated modelling using non-linear finite elements, which we will do later, we will be also considering the cracking and the redistribution that takes place with cracking.

Q- Dr. Mitchell, what precisely are you trying to determine through this 3-D analysis?

A- What we can determine here is the distribution of the loading, particularly on to the cantilever. We can look at the shears and the moments in the cantilever and the variation of the shears and the moments for different loading cases. And so this is important to be able to predict the strength of the bridge.

Q- So, I guess we first turn to page 75.

A- Yes. This is the plan view, top view of the bridge showing the ten beams that you see in this region here, each of them being four feet (4') wide. They are supported on the *assise* or seat of the cantilever. This is the cantilever region here. I'm showing mainly things on the right and in this quadrant where the failure took place.

So, this is the cantilever. This cantilever is supported by an inclined wall that's running along here. In the centre of the bridge is a gap, there is a median here, there is no such gap or expansion joint in the cantilever, it's one solid thick slab across the full width.

I should point out that it's what we call a skued bridge. There is a bias to the bridge. This angle here where the expansion joint crosses over the bridge does not run at ninety degrees (90°) to the edge of the bridge, there is a twenty and a half degree (20 1/2°) angle difference between ninety degrees (90°) and this angle of the expansion joint. So we have an expansion joint on this side and on this side but movement is restricted on the west side of the bridge.

What we also note are the sidewalks along the edge. The sidewalks are quite substantial. They stick out some six feet (6') from the edge of the bridge and they are on the cantilever as well as on the main span. The main span is ninety feet (90') from centre to centre of the neoprene bearing pads.

In addition to that, we see that the supporting abutment has this inclined wall and that has these other additional lateral walls, as you see here, and those lateral walls tend to concentrate the loading effects in the corner. So, if you put a load on the structure somewhere here, we would find that this wall here makes this inclined wall stiffer at this location and more load is attracted to this region so this is the critical region of the cantilever.

Q- And just for the purposes of comprehension, Dr. Mitchell, *la culée*, could you indicate with your sensor where we'll find *la culée*, and I'm deliberately using the French term just to make sure of the comprehension?

A- Yes, well *la culée* is really the abutment that you see here. This is the *porte-à-faux* or cantilever and this is the *trottoir* or sidewalk going along here.

Q- *Et le tablier*?

A- *Tablier* is *ici.*

Q- *Très bien. Merci.*

A- This shows another view of the finite element modelling where you can see the inclined wall and the stiffening walls that frame into the inclined wall and these are the foundations at the base of the walls supporting the cantilever which, in turn, is supporting the ninety foot (90') span, a box girder bridges... box girders which are pretensionned.

When looking...

Q- I'm sorry. And they are called box girders, I should ask why are they called box girders?

A- Well, they're hollow and they are boxed shape and they're, what we call, prestressed so that we can get longer spans.

In the design for nineteen sixty-six (1966) the standard truck was called H20-S16 and it has a total weight of seventy-two thousand pounds (72,000 lbs) or three hundred and twenty kilo-newtons (320 kN). Here you see the wheel loads from the truck. The total truck length is... it can be as small as twenty-eight feet (28'). It varies from twenty-eight feet (28') to forty-four feet (44') and this is the standard truck that one uses for design.

This twenty-eight foot (28') length, if we use the shorter length, is equivalent to eight point five metres (8.5 m). So, this was the designed truck back in nineteen sixty-six (1966).

If we take a look at the two thousand and six (2006) Code, a much heavier truck is used for design. We call it the CL625 because it weighs six hundred and twenty-five kilo-newtons (625 kN) which is almost double the H20-S16. So, this is the truck we use for design today. However, I should point out that although the total load is almost double the H20-S16, the truck is much longer, it's eighteen metres (18 m) long and so it spreads the load out more but nevertheless it would give higher design loads than the H20-S16 truck.

Now, from the finite element analysis, we're able to take a look at the distribution of moments, the bending moments for which we must design and the shears for which we must design, *cisaillement.*

If we take a look at the moment, we can see that the end of the cantilever, the supporting wall is right here.

Q- *Le porte-à-faux*, yes.

A- This is the *porte-à-faux*, yes, absolutely, and this is the inclined supporting wall, this is the sidewalk. And you can see that, from the analysis, the concentration of moment in this particular region, it has a darker colour indicating we have higher moment values per unit width in that particular region.

Q- But it has greater concentration?

A- Greater concentration and we're going to be talking about the concentration effects a little bit later because these are quite important.

Similarly for shear, if you look at the shear, the supporting wall is right here and this is the sidewalk region and these are the locations, the bright areas that you see along the edge, where you see the ten beams on the east... on the south half of the bridge. And they are highly coloured in a bright colour because they are very high local shear stresses there. That's the disturbed region where we have very high stress concentrations but you'll notice that the purple areas in between -- I keep loosing my arrow here -- the purple areas in between these neoprene bearing pad supports indicate that it's a lower stress, very lightly stressed, and similarly, over by the median you see that there is a larger purple area because there is a larger space between the girders here. They are six feet (6') apart rather than four feet (4') apart over the rest of the bridge.

And when we look at the failure later, the failure stopped in this region, starting over in this region and progressing this way and stopped in this lightly stressed region.

Q- Can you come back once more to where -- I will be dealing with this later -- but where the failure started?

A- The failure started right over in this region here.

Q- And on the bottom part, in French we used the terms *efforts tranchants*...

A- *Oui.*

Q- ... and would that be the same as *l'effet de cisaillement*?

A- Exactement, oui.

Q- Très bien.

A- So, this is due to the dead load acting on the bridge and the dead load is quite a large portion of the total loading for this type of bridge.

If we take a look at the analysis that we carried out for the H20-S16 loading, this is the truck loading from the nineteen sixty-six (1966) Code, the standard truck. And if we look at it in the exterior lane, in other words, in this lane right here, closest to the south edge, south-east edge of the bridge, so the truck would be just coming on to this cantilever span creating the stresses that you see. And again we see the concentration of moment and again we see the fact that the shear gets concentrated in this region here because only the outside lane, the exterior lane, is loaded right here, then we don't see as much stress going further away from this region in either flexure or shear because the truck loading is very concentrated on that one lane.

Similarly, if we now take a look at the same truck loading but move it onto the span again but in the centre lane of the bridge, there are three lanes... three lanes across this width here, so we're looking at the truck in the centre lane. But even when it's in the centre lane, we see fairly high moment affects at the point of concentration, as I mentioned before.

And similarly for shear, with the truck located here, we see the high local shear stresses in the region close to the disturbed region and stress is concentrated in this particular region.

Q- How important a role with this analysis served... looked at from the inspection point of view or repair point of view, would that have any importance the fact that there's a concentration of effort at these given areas?

A- Yes, at some point, if someone decided that the bridge should be analyzed in some detail, then this is the way one should do it, one should account for things like the concentration of loads in certain areas and trying to get a better understanding of the structure and certainly with finite elements, it helps us to appreciate what the magnitudes of the stresses are and what the behaviour might be.

Q- And might it serve as a guide also in the inspection procedure itself, would it concentrate one's efforts, certainly at the start of an analysis?

A- Right. Well, the inspection procedure usually comes first, and then if you see a problem then you delph (Ph.) in a more detailed fashion into the analysis of the bridge.

Q- And that's where this type of analysis, the importance of this type of analysis appears, is that correct?

A- Absolutely.
Q- Very well, Dr. Mitchell.
A- So, after having done the analysis to get the loads on the different parts of the cantilever, then what we did is we carried out some calculations to determine the strength of different components in different regions of the cantilever. And when you do this with the two thousand and six (2006) Code, now we're talking about the two thousand and six (2006) Code, and I should mention that we also looked at live loads with the two thousand and six (2006) truck, the CL625, in addition to the H20-S16 truck, because we wanted to check out the bridge design in accordance with the current Code, the two thousand and six (2006) Code.
Q- M'hm.
A- And it uses limit states design where we look at the loads and we factor the loads upwards and we look at the resistances and we factor them downwards with material resistance factors to... and then we compare the factored load with the so-called factored resistance to see if we have adequate safety. That's the procedure we use in the current Code, the two thousand and six (2006) Code.
And just to point out that the calculations with the nineteen sixty-six (1966) Code are made with what we call working stress design, unfactored loads, we look at the

loads at the service load level.

And one thing that needs to be pointed out very strongly is that, in the two thousand and six (2006) Code, there are some significant changes from what we had back in nineteen sixty-six (1966). And just the manner in which we determined if the strength is adequate but certain other important changes as well will be indicated on the next slide.

And one of the major features which affects in particular the analysis of this bridge, in the design of this bridge if you were doing it today, is the scale effect in shear. And I have to take a couple of moments to describe this scale effect or size effect in shear.

A number of different beams were tested and they are shown down near the bottom of this figure by Shioya in Japan and his coworkers, and these beams varied in effective depth from four inches (4") up to eight inches (8"), twenty-four inches (24"), thirty-nine inches (39"), seventy-nine inches (79") and a hundred and eighteen inches (118") or three metres (3 m) deep, that's a very large beam that they tested.

And these series of beams with different depths were all tested by these researchers, and then what they did is compared the shear stress at failure, so in other words, they found the maximum shear on the beam they were testing and they divide that by the area resisting shear of the beam and that gives us the shear stress. And if you divide by the square root of fcprime, you get the non-dimensionalized shear stress.

For this purpose, just think of the vertical access as being the failure shear stress.

And it's compared from these tests with the effective depth of the beam.

Q- So, if I pause for a moment...
A- Sure.
Q- ... failure shear stress, that means at breaking point?
A- That's when it breaks, it ruptures in shear.
Q- And before we move to the second... the bottom line, could you just give us where would it break? Just highlight where it would break, and then we'll come back?
A- Well, one particular test here that was about three feet (3') deep broke here at this shear stress level. Another one that was more shallow broke at a much higher shear stress level. So, what we see from this plot is, as the effective depth of the beam gets larger, the shear stress at failure reduces and this is a phenomenon that wasn't recognized back in the late nineteen sixties (1960's).
Q- When was it first recognized?
A- It was first recognized, I would say, at about nineteen eighty-nine (1989) actually, the test by Shioya, and

other researchers postulated before that, that it might be a significant effect, but in nineteen eighty-nine (1989) they recognized that and we published, Michael Collins, Dr. Collins and myself, published this plot in nineteen ninety-one (1991).

Q- Would I be correct in saying that the larger or the deeper the beam, the less resistant it is to shear?

A- No.

Q- No.

A- It's a bit confusing. If you have a larger beam, it takes a higher shear load but the shear stress at failure goes down because the shear stress is calculated as the failure shear divided by the area. So, if you look at a very big beam, it has a large shear area, right, and it's failing at this shear stress. This beam would have a higher shear capacity than the one on the left but the shear stress is much lower for this case.

Q- So what does that mean in practice, in every day life?

A- It means that things... we realized that some of these larger deep elements might be unsafe and so this is really a startling find and one needs to check members that do not contain stirrups. One has to check them to see if they have adequate capacity, if they were designed before this found its way into the Codes.

Q- Well, since you are on this subject, how would this apply here in Quebec?
A- How would this apply in Quebec?
Q- What would be the impact of the statement you've just made?
A- Well, all thick members without stirrups designed before the two thousand and six (2006) should be checked, that's what it means, just to make sure that we have adequate safety.
Q- So, everyone before two thousand (2000).
A- Right, unless they had taken account of the size effect because Dr. Collins and I had published the designed requirements in the AASHTO Code in nineteen ninety-four (1994).
Q- Donc, on parle des dalles épaisses sans étrier?
A- Dalles épaisses, oui, exactement.
Q- Un peu comme dans notre cas, cette partie du porte-à-faux qui est sans étrier.
A- Oui, exact.
Q- Thank you.
A- And if we look at the significance of this plot, we've shown on this plot the permissable stress, the one point one (1.1) square root or fcprime, if you can recall the discussion we had on Friday, this was the working stress that was permitted -- sorry, I'm having trouble with

this, yes -- this was the working stress level that was permitted, one point one (1.1) times the square root of fcprime, that was the seventy (70) PSI limit that one had in the nineteen sixty-six (1966) Code.

And looking from my calculations on the original design, the original designer had a stress of fifty-eight (58) PSI which is the blue dot on this figure -- sorry, I keep losing my arrow, there we go -- the blue dot on this figure is fifty-eight (58) PSI, so the designer was below the limited seventy (70) PSI but keep in mind that these points here are points at which failure could occur. And this was the safe load limit under service loads back in nineteen sixty-six (1966).

So, that's why it's absolutely imperative that we check thick slabs that were designed according to some of these older Codes.

To determine the resistance in shear for this cantilever, this *porte-à-faux*, the manner in which we calculate the shear resistance is given by this equation here. This is for the case where we don't have any stirrups, which is certainly this case, and we calculate the shear load carried by the concrete acting in tension, there are tensile stresses in the concrete, and forgive the expression here, but in this expression is the same square root of fcprime and there is a beta factor out in front here. So this is really, this front piece here, is the shear stress that one can tolerate at failure multiplied by the shear area. So, shear stress times shear area to get the shear capacity.

And in this expression is this all important factor beta which is reduced as the depth becomes larger, if you have no stirrups or less than minimum stirrups but in our case we have no stirrups.

So, this beta factor gets reduced and we naturally take account then of the size effect in shear.

I should point out that beta is also a function of the stiffness of the longitudinal reinforcement, the main flexural tension reinforcement. As that stiffness goes up, the shear capacity goes up.

So, this is the size effect as treated in the current Code. The conclusion that I had reached was that the resistance in shear was insufficient in terms of satisfying the requirements of S6, two thousand and six (2006) and that wasn't only for the two thousand and six (2006) truck, the CL625 but that was also true for the H20-S16 truck loading. So, we investigated both of these different truck loadings in putting the trucks in different locations and the shear capacity we found was insufficient.

We have a consensus item on this from all of the parties, and it's a very important one, and that is that the design does not satisfy the requirements of S6, two thousand and six (2006) concerning the shear resistance. So, that's an important finding and it's primarily due to the size effect in this case.

Me PIERRE MARC JOHNSON:

Monsieur Couture.

Mr. ARMAND COUTURE:

Q- This statement, Dr. Mitchell, applies even though no stirrups are required in the two thousand and six (2006) Code, is it?

A- Even though no stirrups are required in the nineteen sixty-six (1966) Code, right.

Q- In the two thousand and six (2006), you don't have necessarily to put stirrups according to the present Code?

A- Oh, well, if you... it depends. We were analyzing a bridge that didn't have stirrups and so we've concluded that it doesn't meet the two thousand and six (2006) Code. Now, if you were designing this bridge, you would have a number of things you could do to rectify that problem. One would be to place stirrups, another would be to put in some prestressing, another might be to choose a higher concrete strength but these are all situations that could have been used to remedy it, if you were doing the design today from scratch on such a bridge.

Q- I was more thinking of the adequacy of the two thousand and six (2006) Code in relation to stirrups even though they are not required by calculation, could they be required if the concrete deteriorates?

A- Sorry, it's the nineteen sixty-six (1966) Code that didn't require stirrups. The two thousand and six (2006) Code requires something. Whether you decide as a designer to put in stirrups, prestressing higher concrete strength, it wouldn't have passed the two thousand and six (2006) requirements.

Q- Thank you.

A- This... switching topic slightly and looking at the analysis of what we call a disturbed region, the *zone perturbée*, this is the region, as you recall, near the end of the cantilever close to where we have the neoprene bearing pad and we also have the reaction coming from the hollow box girders that are sitting on these neoprene bearing pads.

And there are really two main resisting mechanisms in this region for lifting the load upwards and that is the *suspentes numéro* 8, the hanger steel number 8, hanger steel which are U shaped, as you can see here, with hooks on the top and the other means is with the inclined number 6 or diagonal number 6 bar which was also in this region. So, I'm looking at the two of them separately just so that we can follow the analysis using *bielles et tirants*, the strut-and-tie model that is in the current Code.

And this is a little bit complex but if one were applying the strut-and-tie model to the U shaped...

Me MICHEL DÉCARY:

Q- Maybe just before you pursue, Dr. Mitchell, it might be appropriate that you remind us of what this model... how this model is to be operated and then proceed in developing in the model which is before us but just if you remind just beforehand.

A- Certainly, this general method for strut-and-tie modelling came out in nineteen eighty-four (1984) in the so-called Building Code in Canada and in nineteen ninety-four (1994) it appeared in the AASHTO Code, the American Bridge Code and what it does is it's a way of modelling the flow of the forces through a very complex region which is certainly what we have at the end of this cantilever. It's a very complex region.

And the way in which you visualize it, I always start by visualizing the way that compression would flow into the concrete and the compressive struts or *bielles* are shown with the dashed red lines.

So, I always start with the compression, see where I think it should go. I want this hanger steel to pick up the load here and there should be another strut coming over to this other leg of the hanger steel. It seems to be missing but there should be another red dashed line here to lift this load up as well. And then...

Q- But there was none in this case.

A- I'm sorry?

Q- There was not, this additional bar in this particular case?

A- No, I'm talking about the fact that I should have another red dashed line here.

Q- Yes, I understand.

A- And I'm going to put it in for you right now. So, we should have... it's supposed to be a straight line. There should be another dashed line coming down to there.

Q- It compares to mine, Dr. Mitchell.

A- And once you start building up the strut-and-tie model, you see where tension is required, at least this is how I proceed. In other words, this compressive strut thrusting outwards at this node of the thrust needs to be pulled back by some tension reinforcement. So, we model the strut-and-tie model with the dashed lines which are the compressive struts and the solid lines which are the tension ties or *tirants*.

And it's very complex because this bar, for example, which is in tension, needs to be anchored somewhere and that requires some compressive struts to anchor that bar, as you can see there, there is a fanning compression anchoring this bar.

Q- I'm sorry, I can't see where you're at. Maybe other people can and I wish to apologize.

A- Let me...

Q- I see.

A- Let me back to the arrow.

Q- Yes.

A- So, this is the tension tie which needs to be anchored somewhere over here and so to anchor it we need compressive struts fanning out like this, putting further loads into the first leg of the *suspente* or hanger bar and the second leg as well.

And when you get to the top here, then the compression struts go over and create shear in this region of the beam so these two struts represent fanning compressive stresses which are creating beyond this point a regular shear zone or beam zone as we've seen previously.

And furthermore, these compressive struts that you see pushing on this node needs some tension to hold the outward thrust back and those are the hooks of the number 8's. And unfortunately, the hooks aren't sufficient enough to pull this force back and so we have to start to use the number 14 bar to help pull the force back and I'll come back to that in a minute.

The number 6 diagonal bar, the strut-and-tie model is a little easier to understand, this strut comes down. We have another compressive strut from the bottom and we have a tension tied to put this node in equilibrium, coming up like this, and then there is compression that flows into the rest of the beam.

Again, the number 6 hook is not sufficient enough to develop the loads we need and we find that this strut will also seek out the top number 14 bar and this pushing against the top number 14 bar, because there is no steel between the number 14 and the hooks, creates tension between the number 14 bars and the hooks, sorry, between the number 14 bars and the hooks.

Q- M'hm.

A- And similarly, if we go back to the more complex one, here I have shown that extra strut that was missing on the previous one, but I should mention that, in the case of the hanger reinforcement, the hooks are insufficient to develop the required tension and, once again, these compressive struts seek out the number 14 to anchor on too and that creates tension in the zone between the number 14's and the number 8 hooks.

Q- Could we come back to...

A- Sure.

Q- ... page 91? Now, at the *suspente ou barre en étrier*, the *suspente*, the red bars here, had they been tied to number 14 bars? Both of them, would have been this *zone de tension*?

A- Very good point. No, if the... I think we discussed it on Friday the sort of detail where you really should hook the number 8's, turn them around and hook the number 8 bars around the number 14 bars and similarly on the bottom, hook the number 8's around the bottom reinforcing bars here to give a very strong link over the entire depth of the member, top steel to bottom steel, then, you would have reinforcement crossing this zone, you would not have that zone of tension, that would be much better.

Q- Very well. Now, the next analysis is by non-linear finite elements. Could you describe this type of analysis?

A- Certainly, I will just put the first slide on. This is an in-depth study of the disturbed region using finite elements, as we have used before to look at the entire picture on the bridge, but this time non-linear finite elements because we track the cracking in the concrete and we track the stresses in the reinforcing steel that become highest, right at the crack and we are able to check against slip of the cracks, which would donate a shear failure.

And I will be talking about the non-linear finite element modelling a little bit later in more depth as well, but in any case, it accounts for the changing stiffness, as you load the member and his cracks develop and in this particular case, what I wanted to point out is the flow of stresses.

These are what we call the principle compressive stresses, this is the bearing pad here with a vertical load and what is really going on, we see that because there is tensile strength in the concrete...

Q- I am sorry, just a reminder. What is tensile strength?

A- Tensile strength is the fact that it has some resistance to tension, but not very much, and when it cracks, hopefully there is steel crossing that crack to transfer the load across, across the crack. So, here is the neoprene bearing pad, we have a very large vertical load and we see that the compressive stresses are actually curving in here.

Now, as a simplification in design, we assume that it is a straight line compressive strut, it is a simplification and usually gives conservative designs, but I needed to know exactly why it could sustain higher loads because the simple strut-and-tie model said we were in really deep trouble, but we know there are some tensile stresses in the concrete, which we normally do not account for in design.

But here, we see the curving compressive stresses, we see the compressive stresses coming from the end of this bar going up, and you can see the zone between the number 14's and the number 8 hooks and you see the compressive stresses pushing against the number 14 bar.

So, those white lines, those faint white lines are compressive stresses pushing against that number 14 bar and to see the significance of it, if we take a look at the next slide, these are the tensile stresses, in other words the stress in tension in the concrete.

And we see that there are tensile stresses, which are actually going, of course, perpendicular to the principal compressive stresses, they are slanted the other way and, therefore, you can see that the zone between the number 14 bars and the number 8 hooks is a zone of tension and high bond stress.

So, there is bond stress and there are tensile stresses, shear stress from the bond, tensile stresses as well, and eventually, cracking occurs in this particular region with the cracks oriented in this direction.

Q- So, the cracking is diagonal?

A- The cracking is slightly diagonal.

Q- And would that be characteristic of some type of particular type of cracking?

A- Yes, bond-type cracking, but we also have tension in that zone, as well...

Q- M'hm.

A- ... from this detail. And this shows the predicted stresses in the reinforcement in that region, again using the non-linear finite element analysis, and you can see that there seems to be a hot spot right here, where the number 8 bars are highly stressed and the diagonal number 6 bars is quite highly stressed, but nowhere near the yield stress.

So, they are not yielding, but there is a crack that comes right here through this region due to the loading and that stresses the reinforcement in this region.

Q- Can you define the yield stress?

A- The yield stress, in this particular case, for the real bridge is forty thousand (40 000) PSI, two hundred and seventy-six (276) MPa.

Q- And what does yield stress relate to?
A- Right, if you were to take a reinforcing bar and pull on it, which is really its function, resisting tension when we design a reinforce concrete structure, so if you pull on a bar, it has quite a large stiffness until which point we reach the yield stress, and then it deforms quite largely and that is what we call the yield point.
Q- And do you say that again in that area, an area with other types of efforts, there is also quite a bit of yield stress on the bars?
A- These are not yielding here at this location, but the number 6 bar that we placed in here to simulate the actual construction shows some bit of yielding.
Q- I see.
A- Okay. These are not yielding, the number 14 bars are picking up more stress as you go more into the span of the cantilever, these are more highly stressed over here. So, from using such a model, you can start to appreciate the behaviour in a little more detail and see where the steel should be stressed the most and where your critical regions are in the concrete.

The conclusions that we reach from this analysis using the two thousand six (2006) Code, and here we are talking about the strut-and-tie model because the non-linear finite element method is a more sophisticated model that goes a little bit beyond the Code requirements, but from the Code requirements themselves using strut-and-ties, we see that the *suspentes* or the hanger reinforcement anchorage is inadequate and the same thing holds for the number 14 bars, the anchorage is inadequate.

And we talked on Friday that those bars should have been hooked or somehow anchored at their ends to develop higher stress. And in addition to that, in the regular beam zone, the shear resistance is insufficient to resist the shear or *cisaillement*.

Q- Had there been, dans la zone régulière de l'armature des étriers, would that have changed your conclusion?

A- It would have changed the second one.

Q- Yes?

A- Absolutely, yes. Yes, I have checked that out actually and it does make a difference, if even minimum shear reinforcement were placed, then it would have passed the Code.

Those were our conclusions from our team and there is, however, a consensus on the detail of the steel and we concluded from all the parties that the designer, the manner in which the designer anchored the *suspente en U*, the hanger reinforcement, the number 8 U-bars near the top of the cantilever, at the end of the cantilever, does not respect or does not meet the requirements of the two thousand and six (2006) Code.

Q- But just to remind everyone, it has been established, we've established beforehand that the designer did respect nineteen sixty-six (1966) Code.

A- Absolutely.

Q- This is always in regard to the two thousand and six (2006) Code.

A- Absolutely, yes, and you know, the designer should not have to know what Code changes are going to take place in the future. So, the designer was designing in accordance with the sixty-six ('66) Code.

Q- M'hm.

Me PIERRE MARC JOHNSON :

Q- Could you say, Dr. Mitchell, that the design respected the best practices at the same of nineteen sixties (1960's)?

A- Yes, that is a more difficult one to answer and I think Mr. Nicolet was asking that question as well last week. But we have looked at the literature that was available, at best there is one rather obscure book that has a detail that is somewhat better than the one chosen, but I have a lot of difficulty with that detail as well, that was published in nineteen sixty-seven (1967), that book, published out of New York.

But also, we have looked at a number of bridges which were similar and built in the province, the Joliette bridge and the St-Alphonse bridge were built around the same period of time and they were designed by an entirely different designer and they had details which were deficient as well.

So, I have looked at those details and studied them and there were problems with those details and one of those bridges has been since closed, the other one has been torn down, principally because of those details and the fact that there could be a shear problem as well.

Me PIERRE MARC JOHNSON :

Monsieur Couture.

Mr ARMAND COUTURE :

Q- Could you, Dr. Mitchell, come back to page 95?
A- Certainly.
Q- Should the suspense bars have been torn at the same level as the number 14 bars, the three horizontal sections of the suspended diagonal bars, if they had been at the same level as the number 14, would your conclusion be the same?

A- If they had been at the same level as the number 14 bar, they still are not anchored around the bar and it creates a slightly different problem, as you will see when we get to the test that we carried out, that if these number 8 hooks were at the same level as the number 14 bars, we get a significant amount of congestion of the steel at that level and bonds start to play, bond failures start to play a more important role.

So, it wouldn't have satisfied the Code and it wouldn't be good practice today.

Q- Thank you.

Me MICHEL DÉCARY :

Q- But again, just to follow up, because we are going to address this a bit later, but just to follow up on Mr. Couture's question, we know that the designer had designed the number 8 bars to be parallel to the number 14 bar and again, it goes back to Mr. Couture's point, is that doing this, he was not designing against the provisions of the nineteen sixty-six (1966) Code.

A- No, the designer was trying to anchor that reinforcement with hooks at the top at the same level as the number 14 bars and he was trying to give a lap splice between the number 8 hooks and the number 14 bars and he also put on longer hooks, being one and a half (1 1/2) times longer than the Code required.

So, there was an attempt by the designer to do what he thought was correct.

Q- And again, just to make sure that we come full circle based on questions by the Commission members, was that against to have these 8 and 14 bars running parallel instead of being hooked, would that be against good practice or best practice in nineteen sixty-nine (1969)?

A- It certainly would be today, but back in nineteen sixty-six (1966), there are some sketches and other structures that have worst details and so, I do not think that the designer could be blamed for any of those types of deficiencies.

Q- Very well.

A- Just to go on with some of the consensus items regarding the material I have just presented, not only were the number 8's inadequately anchored, those of the vertical suspension bars, but the number 14 longitudinal bars did not conform to actual practice one would like to normally anchor those bars by hooks at the ends. So, we have consensus on that detail as well.

Q- On this one, I personally have... my problem is the following for those that followed the Commission, many workers or a few workers who testified, I was under the impression that it was quite well known that if you had a bar, a vertical bar going up to reach a top bar, a parallel bar, that everyone had the reflex or would naturally come to the conclusion that it would have to be hooked in order for it to be effective.

I thought that was quite well known in nineteen sixty-nine (1969), nineteen seventy (1970). Am I wrong?

A- You are absolutely right concerning stirrups, stirrups we always hook them around the top and bottom reinforcement, and so that is standard practice. Back in nineteen sixty-six (1966), however, they had no design requirements or no standard details for disturbed regions and that is the difficulty.

Q- But there, the second point with this, using the term "stirrups", *étriers* en français...

A- Oui.

Q- So, lorsqu'on parle what you now call... when I first started questioning various witnesses, I referred to these *étriers en U* as stirrups, *des étriers en U*, and again, it was in relation to this questioning that the thought was that they had to be connected to the top.

Now, because it is a disturbed area, in your analysis I noted that you changed the terms. So, instead of referring to stirrups, now you refer to them as... in English you might refresh, how do you call this?

A- Hanger bars.

Q- The hanger bars, but how does it change your reasoning?
A- Well, it does not change my reasoning one little bit, but...
Q- No, but...
A- ... no, I see what you are getting at, I mean...
Q- I did not think it would change your reasoning, Dr. Mitchell.
A- But you know, what I think is most difficult for any engineers that would be watching, listening in, is to be able to put yourself back in nineteen sixty-nine (1969) and try to decide what one would have done. I mean a name is just a name, we call them hanger bars, we call the stirrups, in the beam region stirrups, stirrups are normally anchored around the longitudinal reinforcement.
     Hanger bars was a little bit unknown back then and the other details I have seen on other bridges, they have lousy details as well. So, it seems to be bad practice for the design of those regions from a number of different companies in Quebec. I have not looked elsewhere, but in Quebec.
Q- Very well.

Mr. ROGER NICOLET :

Q- Excuse me. All this begs the question, but in all the Quebec bridges, you have looked at, have you found, dating back to this period, have you found some good details?

A- Well, this is a rather unique beam seed area and I know that the MTQ has been looking at all bridges with beam seeds and they would be better able to answer what they found. But the two other examples that we looked at had poor details, were much worse than the details we see on this bridge, and one of those is torn down, the other one is supported.

Me MICHEL DÉCARY :

Q- Just a comment on Quebec and I do not want you to go too far, but is there any likelihood that the same type of problem may exist elsewhere?

A- Outside of Quebec?

Q- That is because the Codes were silent on this, there was nothing in the Code to cover this and as you pointed out, best practices, there was very little on this anywhere in the world. So, it was left to the initiative of each designer to try to find his way, if I may use that expression.

So, do you want to expand on this last comment in respect to Quebec as being, you know, obviously focusing on Quebec, but is it not likely that it goes beyond the province of Quebec?

A- Absolutely, it is a North America problem, I would say, Canada and the US and there seems to be some differences in Europe with regard to design of these regions. I think they were much more tuned to trust models and there were some really good detailing practices, particularly in Germany and I think anybody that has been exposed to the German experience would appreciate all of the details that were coming out of Germany at that time.

But I have not seen any details generally published that met the mark in nineteen sixty-nine (1969).

Q- Thank you. The next chapter I think we should call upon, Madame Desprez, pour l'ajout numéro 1. Am I going ahead of time?

A- No, that is perfect and there were two slides that were left out of my presentation, but they have already been shown by Dr. Marchand, so there is no surprises here, but just to make the story complete, I wanted to show the shear failure on the bridge and it is clear, you can see that, I am trying to get my mouse or it is maybe a different system.

Okay, it does not matter if you, no, it does not work, okay. The shear failure, if you take a look at it, there is a curved shape going in towards the incline wall support at the back of the photograph, you can see that the shear failure has taken place just right over the hooks of the number 8 *suspentes* and the number 6 diagonal bars.

It has just gone right through that weak plane and this is very important, that is the weak zone we have been talking about. I wonder if we could just zoom in in the upper portion of that picture. No? Okay.

Q- Maybe... is it possible to use your cursor, arrow, yes, good.

A- I am not moving it, okay. If we just go up a little bit with the arrow and to the left, à gauche un peu, well it is going to be difficult, but if you stare at the picture, you will see towards the back of the failure surface some number 6 bars poking out of the bottom of the failure surface, right near the back edge of the failure surface.

And these number 6 bars, we believe were placed to help the construction and support the top reinforcement during construction. So, that is all I wanted to mention for that, if we go back to the...

Me PIERRE MARC JOHNSON :

Q- But they were not on the plans?

A- They were not on the plans and I will speak more directly about them later, when I can manoeuvre my arrow on the screen.

Q- Correct.

A- So, having shown you the shear failure, I think you would appreciate the consensus that was reached by all the experts on the mode of failure and it was by rupture in shear in the cantilever on the south-east side. Now, this comes to the next part of my presentation talking about the test that we carried out at McGill University, looking at cyclic loading of some slices, if you like, of the *porte-à-faux.*

Me MICHEL DÉCARY :

Q- I am sorry, would you mind coming back to page 100?

A- Sure.

Q- And you may want to just illustrate, we saw the picture, madam Desprez showed us the picture, but you may want to just illustrate, possibly by using your hand just in front of the camera, how this *cisaillement*, you know, along *la zone de faiblesse* operated. Just one moment, because we are coming back to it, but I think because we just finished that part, the *cisaillement*, how it was ruptured, comment ça a été déchiré?

A- Yes, yes, okay. Well, that is very hard to do, but basically, if I show you with a piece of paper, we have, if you visualize this being the end of the cantilever and we had a horizontal crack coming along the level of the number 8 hooks...

Q- Yes.

A- ... and the number 6 hooks, and when it got to the end of those hooks, it starts coming down like this, and then comes along in this direction. So, that is the shape of the failure. Okay?

Q- Yes.

A- And what I was trying to illustrate earlier was the fact...

Me PIERRE MARC JOHNSON :

Q- You have interesting pastimes.

A- That is as good as it gets, yes. And what I was trying to illustrate to you earlier is that on this shear plane, there were some number 6 bars interconnecting or playing together the bar.

Me MICHEL DÉCARY :

Q- Maybe just to mark the other side so we can see where the number 6 bars were, just by using your fingers on the other side so we can all see it.

A- Certainly.

Q- Yes, very well, thank you.

A- Okay. There we are coming down and crossing the plane right here and they did not have a tremendous effect, but they were there, they were placed about every four to five feet (5') across the width of the cantilever.

Q- Very well. Thank you, Dr. Mitchell.

A- So, looking at the cyclic loading test that we carried out at McGill, the objective of these tests was to study the behaviour of the cantilever with different details. At one end... we had a double-sided cantilever as you will see, at one end, it was "as designed" details, as we figured the designer had meant from the structural drawings and, at the other end, it had the "as constructed" details.

So, we would be able to compare one for one what the difference would be. I should point out that in our test, we only had a few months to do the test, so we do not have any degradation of the material, we have perfectly a healthy concrete in our test specimen.

That is an important difference between our test and what happened on the actual structure. Nevertheless, carrying out this test helps us to understand the failure modes and the behaviour of the cantilever. The steel that we used were all imperial size bars that we imported from the United States, simply because we wanted to use the actual bar sizes and the information that would have been used on the actual bridge.

Q- And can you define so that these tests being carried out on a prototype, how would you define *essais de chargement cyclique*?

A- Yes. What we did, if I show you the next slide, is that what we did is we built a double cantilever. So, here is the *porte-à-faux* on one side and here is the *porte-à-faux* on the other side, the cantilever on the other side. What we are going to be doing is loading the neoprene bearing pads, bringing it up to the dead load and then cycling an additional live load, representing the truck on this and repeating the loading much as like what you would have on the actual bridge.

So, what we have done is we have recreated a four-foot (4') thick slice, if you like, of the cantilever and the total span from the face of the support here to the centre of the bearing is thirteen feet (13'), much like the span we have in the actual structure.

We've tried to simulate all of the details of the reinforcement, at the south end, it's the "as built" details, which I will show you in just a minute, and at the north end the "as constructed" details.

This shows the cycling in a little more detail that you were asking about, it is a double-sided cantilever and we have a huge steel distribution beam and we supply the load to the top of that beam, so we get equal loads at both ends of the cantilever and we put a dead load of three hundred and fifty kilo-newtons (350 kN) on each end and then we cycled the live load representing the truck from zero to ninety kilo-newtons (90 kN) for almost nineteen thousand (19,000) cycles and why we were doing it that way was and why we were doing it that way was to try to represent the actual traffic that was on the bridge between nineteen seventy (1970), when it was built, and nineteen ninety-two (1992) when the expansion joint was replaced.

So, we wanted to have realistic loads, we wanted to have realistic cycles and a report was carried out by Mr. Hamaoui who looked at the likely number of cycles in that particular period of this... for the bridge and what the likely loading might be.

And so, we used that data to try to simulate the loading up until we replaced the expansion joint.

Q- I'd like to pause a moment and just remind everyone that the report prepared by monsieur Hamaoui was remitted to all parties and was produced, and everyone agreed that it could be produced and it was unnecessary for Mr. Hamaoui to be examined.

A- The next slide shows the test set up in the laboratory at McGill University, you can hopefully clearly see the cantilever on the south side, you can see the large steel loading distribution beam that we used, we have a computer controlled machine capable of producing cyclic loading and, as we load in the centre here there's a ball bearing, a big ball joint, I should say, in the middle so we end up with equal loads at both ends, insuring that both ends are loaded identically during the testing, had the same history, identical loading.

And one other thing is that we cast these two cantilevers at the same time, it took two batches of concrete, but they have the same concrete as well on each cantilever.

Q- What is "cyclic"?

A- Cyclic means you're repeating the load from zero up to some magnitude and releasing it and repeating that in a cyclic fashion.

Q- Thank you.

A- Underneath is steel column, we have the neoprene bearing pads we've used actually, the actual bearing pads from the evidence site, so we've used the identical bearings used on the bridge.

This next slide shows the details as designed from... that we obtained from the structural drawings and you can see the green, in green, the U bars, number 8 U bars, the pink are the number... number 10, *épingle* or U bars in the *assise* or beam seat area; the yellow represents the diagonal number 6 bar, that was on the drawings, these are all going around number 7 transverse bars; the red bars are the number 14 bars at six inch (6") spacing. The *suspentes* are shown here, their number 8's, at ten inches (10"). The U bars, the pink U bars are number 10's at five inches (5") and we have on the bottom number 8 bars and again number 7, transverse bars.

Q- Dr. Michell, you said this is the as-designed and the design, the plans used, were those remitted to you by monsieur Dupaul...

A- Yes, they were.

Q- ... the designer...

A- Yes, they were.

Q- ... before plans?

A- Absolutely.

Q- And on this one the as-designed, because we'll get to the as-built, the number 14 and number 8 bars are parallel, have been set parallel one to another, is that correct? Even though you might see from the plan that there's red above the green?
A- Well, no, actually the number 14 has a larger diameter...
Q- Oh, that's why, I'm sorry.
A- ... than the number 8.
Q- Thank you.
A- That's why you see a bit of the number 14 bar above it, yes.
Q- Thank you. And they are parallel.
A- Yes, they are parallel and the number 8 hooks are hooked around number 7 bars in the corner, which are right underneath the number 14 bars.

Also over, near where the wall support is, which were represented by this reaction area here, we have the inclined reinforcement coming out of the wall, we just wanted to represent that steel as well.

And, in order to determine what the details were for the as-built end of our test, we had to do some dissection at the evidence site to be able to determine what the exact details were in the region of interest, which is near the failure, and we were able to uncover the steel and I'll show you some more details of it a bit later, but the number 8 U bars are shown here, this hook has been cut off, a sample had been taken to get the stress strain relationship, the get the yield stress of that steel to be able to identify it a little bit better.

This is the number 6 bar that had to be obviously forced into place because of the difficulty of placing that steel, this is the number 10 U bar, and the next line here shows a clear picture of a slice we took later exposing the steel, and you can see clearly the number 8 U bars with the hooks, the number 6, diagonal reinforcement and the fact that this angle here had to be squashed a little bit to force the number 6 diagonal bar into the right location, and these are the number 10 U bars.

And what it looks like is shown on the bottom right here, that's the location of the reinforcement and what we found is that the bottom most bars were supported on number 6 crossbars during construction and this supported the number 8 bottom longitudinal bars, and the number 10 *épingle* or U bar, but also the number 8 *suspentes* or hanger bars.

And, because the bottom surface is sloping, because the cover is constant, this meant that the bottom of the number 8 U bars are parallel to the bottom surface and this puts them at a slight angle instead of being vertical.

So, they're at a slight angle and, furthermore, because they were placed this way, they were placed downwards from the position that the designer had intended.

So, the hooks of the number 8's and the hooks of the number 6's come somewhat below the number 14 bars as was described in Dr. Marchand's testimony.

We confirmed that the cover concrete was two inches (2"), this is one of the number 6 supporting bars here, but the cover was two inches (2"), as we suspected. So, we had to determine this information before we could build the as-constructed side.

If I just go back one, there was something I wanted to mention. We found as well number 6 bars shown here not only at the location further back towards the wall, as I showed you previously, but also up near this region and we surmised that this number 6 bar, which was welded to this number 6 crossbar here was probably used to support a number 7 crossbar to hold up the top steel during construction, and that's shown on the figure on the bottom right here where we have the number 6 bar going upwards.

Q- And that bar, obviously, did not appear on a design, on a plan?

A- This was only placed during construction and these bars were not on the plans. And, again, they were about four (4'), five foot (5') spacing across the width.

Q- Now, Dr. Mitchell, just one point, my understanding from the proof that was established before the Commission was that these bars may have been inserted, this number 6 may have been inserted without the designer knowing about it. Do you have any comments on that? And Mr. Dupaul testified that, in effect, you know, these were not authorized by him and it was, I understand, practice often to add certain bars at various places, you know, to help during the construction, to hold thing together, now...

A- Yes.

Q- ... do you have any comments on that?

A- No, that's fairly common practice that that top layer of steel had to be supported somehow and this was the method that the contractor chose.

And we have a consensus item, and it's a rather obvious one here after having seen everything else, that the steel in the upper area near the extremity or end of the cantilever were badly placed, in particular the number 8 U hanger bars and the number 6 diagonal bars.

Q- Ce qui créait la zone de tension, la zone de faiblesse.

A- *La zone de faiblesse*, the zone of weakness, absolutely.

During the construction of our test specimen, we wanted to match identically the details that were on site and so here is the number 8 hanger bar that I showed you inside the concrete on a previous slide, where this portion had been cut off to take a bar sample.

We did however see the imprint of the end of the bar in the concrete and were able to mark exactly were this hook came to so that when we received our number 8 bars from the United States, to get the same, exactly the same size, we were able to match identically the details of the hooks, the overall height, etc., so they match identically.

And this is just a drawing which shows the details of the reinforcement in our north end, which is the end with the as-constructed details, showing much of the details required to appreciate and build the specimen.

So, here is the additional number 6 bar. Here we see the corner angle and reinforcement which I'll show you a little bit later for the expansion joint area.

Naturally...

Mr. ARMAND COUTURE :

Q- Excuse me.

Me MICHEL DÉCARY :

Q- Dr. Mitchell.

Mr. ARMAND COUTURE :

Q- ... Dr. Mitchell, do I read this correctly that the vertical number 6 bars additional were placed only on the as-built side and not on the as-designed side of the test specimen?

A- That's correct. That's absolutely correct. The other side didn't have any of those vertical number 6 bars. Right. Thank you. Naturally we tested all of the reinforcing bars with three test coupons to be able to determine what the properties of the number 8's, the number 7's and the number 6's were, as well as the number 14 and number 10 bars which were used to construct the test specimen.

I'm just going to show you fairly quickly some photographs of the reinforcement in the formwork before we cast the concrete during construction just so everyone can appreciate the amount of reinforcement in this very complex region.

So here we see the as-designed and with the details we see the number 8 hanger bars, we see the number 10 U bars, we see the number 7 crossbars with the hooks going around the number 7 bars.

At the north end, similar details, we have strain gauges on the steel, these measure the straining of the steel or the elongation of the steel so we can tell how much the steel is working at each of these locations were we've glued the gauges.

This shows the details of the hooks on the as-designed side with the number 8 hooks here and here and the number 6 hooks for the inclined reinforcement.

This is the north end and, coming up, you can see that the hanger bars are going below the number 7's this time, as we found on the plans, and an additional number 7 was placed in the corner, as we found in the field at the evidence site.

So, this is what the details of the hooks looked like on the as-constructed or as-built side with the number 8 hooks going below the number 7 bars, and you see that these become more and more inclined as you go further out because, of course, the vertical legs of the *suspente* are not actually vertical, they're slightly, slightly inclined.

This is after placing the number 7 transverse bars and the number 14 longitudinal bars, you see that there's quite a bit of reinforcement in here.

This shows the detail of the expansion joint reinforcement which consisted of thickening of the concrete of two and a half inches (2½"), steel angle placed on the outer edge and, welded to that steel angle, were steel straps bent at their ends to hanker the steel angle to the concrete.

This shows the details of the reinforcement on the as-designed side and really you can see the amount of congestion of the steel here. The number 14 bars, the number 8 hooks and the number 6 hooks, there's a lot of steel across this particular plane.

This is the other end, this is the as-built end, we used the same detail for the... for the original detail for the expansion joint reinforcement and this is what it would like looking down on the top of the as-built end, you can see that the number 8 hooks are coming out below the number 14 bars, and missing the number 7 bars over in this region.

Me MICHEL DÉCARY :

Q- If I pause for a moment, at the end of the construction, just before people were like called in to put the concrete, there's supposed to be en engineer or someone from the engineering side that takes a look at the system, the structure, and is supposed to say yes or no before there is any concrete being poured.

Isn't it apparent, if you have the plan and you see the number 8's in particular, because those are so obvious - to me anyway - that they're not in line with not only number 14, but they're below number 7.

And in those days, people realized that the things to operate had to be hooked together, I mean, they had to be hooked at one point, they had to be over number 7. Do you have any sense of why anyone would have authorized this?

A- Well, I can't quite answer that question, but certainly upon looking at this, it's deficient and it doesn't match, it doesn't match the plans or the drawings and so a phone call to the designer would have been appropriate...

Q- And...

A- ... in halting the construction at that stage.

Q- I was trying to find an explanation. Can you find an explanation as to why... is it any reason, in the engineering world, any reason why someone could accept this, that's unbeknown to us, or is that just not in accordance with plan and, you know, as he said, at least it should have given a designer a ring?

A- Absolutely it should have, you know, position of hooks, anchorage of steel is vital and something should have been done, yes.

Me PIERRE MARC JOHNSON :

Madame Desprez, est-ce que c'est possible de retrouver la photographie prise au moment des travaux effectués en quatre-vingt-quatorze ('94), est-ce que vous avez la cote?

Me MICHEL DÉCARY :

Oui, on va y arriver en quatre-vingt-douze ('92)? Vous parlez de la réparation...

Me PIERRE MARC JOHNSON :

Pardon, en quatre-vingt-douze ('92).

Me MICHEL DÉCARY :

C'est très bien, oui, je comprends. Monsieur le président, si vous n'avez pas d'objection, nous allons adresser cette époque en détail et, justement, qu'est-ce qu'on voit avec la fenêtre d'observation et les personnes qui étaient là, et je pense que ça va faire l'objet de commentaires du Dr. Mitchell...

Me PIERRE MARC JOHNSON :

Merci.

Me MICHEL DÉCARY :

... précisément sur ce point, mais si vous y tenez, on peut bien...

Me PIERRE MARC JOHNSON :

Non, non, ça va. Mais, en fait, c'est parce que vous soulevé le fait qu'il est manifeste, là, que dans cette reconstitution du tel que construit, un certain nombre de barres qui devaient être ou parallèles...

Me MICHEL DÉCARY :

Oui.

Me PIERRE MARC JOHNSON :

... au 14 ou, au minimum, en tout cas, par-dessus, les numéros 7, ne le sont pas.

Me MICHEL DÉCARY :

Oui, oui.

Me PIERRE MARC JOHNSON :

Et ça, on le constate au moment de la reconstitution au laboratoire de l'Université McGill et on peut penser que ça avait l'air de ça quand le surveillant est venu ou a peut-être oublié de venir...

Me MICHEL DÉCARY :

Dans ce cas-là...

Me PIERRE MARC JOHNSON :

... et on peut penser que c'est aussi de quoi ça avait l'air quand, en mil neuf cent quatre-vingt-douze (1992)...

Me MICHEL DÉCARY :

Voilà. Page 173, Monsieur le président.

Me PIERRE MARC JOHNSON :

Oui.

Me MICHEL DÉCARY :

Q- So...

Je m'excuse, je vous ai interrompu.

So, Dr. Mitchell, see where Mr. Johnson is leading to, nineteen ninety-two (1992), repair work is done and this is what the people involved in the repair see and, in essence, is that the same as we've just seen in the as built?

A- No, I think we've pick the wrong location...

Q- The wrong...

A- ... because in this case one can see that at least it looks like the hooks are the same level as in number 14...

Q- Yes.

A- ... which was actually reported on one corner of the structure, but there are better photographs - and I'll be coming to some later, actually.

Me PIERRE MARC JOHNSON :

Q- O.K., ça va.

Me MICHEL DÉCARY :

Q- So, we can come back to that, because it was not the...

A- Okay. Just to indicate that we added the number 6 bar in what we thought was approximately the right location, it seemed to vary a little bit across the width of the structure but we've added it in on the as-built side and it doesn't exist on the as-designed side.

Me PIERRE MARC JOHNSON :

Q- Can you explain to me again how these number 6 wound up there in your opinion?

A- Yes, if I can go back, let me just choose the right photograph for you here.

Page 108, s'il vous plaît.

Me MICHEL DÉCARY :
Q- Yes.
A- Oui, merci.

This diagram shows it, the number 6 we believe were there to help support the top reinforcement so the number 6 was welded to this bar which was support at the bottom, it comes up and it was probably we think welded to a number 6 at this level to hold up the number 14's, during construction.

Me PIERRE MARC JOHNSON :
Q- What would hold it up on number 6 is what, three quarter inches (3/4")?
A- Right.
Q- So it would hold up the 14's on a quarter inch (¼")?
A- Well, I suspect that there was some weight transferred through this part as well.
Q- Okay, thank you. That's fine. Better than nothing.
A- Right. Well, if you recall the ground penetrating radar readings we had, we saw that the top number 14 bars were quite wavy and where they were the highest is where we think we had the number 6 vertical bars and in between they were sagging.

So, it fits with what we've seen on the GPR readings I've taken on the structure.

Q- Thank you.

A- So, that's the number 6 bar that we placed in the construction in the laboratory, and this is the additional number 6, there were two of them in the as-built side of the cantilever.

And here you could see the as-built side with the two number 6's, and this just shows where we've placed some strain gauges on the reinforcement to measure the strains and the flexural reinforcement strains and the number 8 U bars and the number 10 U-shaped bars and the number 6 diagonal bars.

And similarly on the as-designed side, and you'll note that there's no vertical number 6's on the as-designed side, and the gauges were placed in the same locations on this side.

In addition to that, we have some devices that measure change of length between two pins that we drill into the concrete and this enables us, if we get a crack forming for example, then we would see change in length in the region were the crack is crossing one of these gauges and so, by doing that, we can follow the behaviour very closely and this shows the linear voltage differential transformers that are measuring the change in length between the top and the bottom points here and similarly all the way along the length of both ends of the cantilever.

We're also looking to see what the effect of this crack that forms on what we call a re-entrant corner by the *assise* here, we were picking that movement up with this gauge as well.

Me PIERRE MARC JOHNSON :

Q- Is this life size in terms of the *chaise* part?
A- Absolutely. Absolutely, we've... this has the same dimensions as the actual *porte-à-faux* or cantilever. The only thing we've done is we've taken a four foot (4") wide slice of it.
Q- Yes.
A- And... but everything else is identical dimensions including actually using the same neoprene bearing pads.
Q- Good.
A- Now, looking at the test, what I want to show you first is how the failure occurred at the south end of the specimen, but before we look at it in a little more detail, I just wanted to point out that the failure occurred when we had a load at the end here of eight hundred and ten kilo-newtons (810 kN).

This is the cracking pattern that you see, then we're very close to failure, within a few per cent of failure, about five per cent (5%) of failure, ninety-five per cent

(95%) of the failure load.

I just wanted to point out the flexural cracks that you see on the test specimen and something that was also very interesting, these horizontal cracks that you see at the level of the number 14 bars, those horizontal cracks at the level of the number 14 bars were there before we started testing. We believe they were shrinkage cracks because there was a congestion of steel across the widths and the concrete shrinks more on the top than it does on the bottom, and we end up getting shrinkage cracks around the bars.

But they seemed to have healed a little bit through what we call autogenous healing and they didn't open up until later on in the testing when we increased the load.

Nos, what we saw was a flexural crack at this location and we were keeping our eyes on this crack, this inclined crack that was forming, and as soon as that crack joined up with this flexural crack, we knew that this was going to be the critical shear crack that was forming.

So, we were watching this area very closely. Please, note that we're very closed to failure and the crack width is zero point two millimetres (0.2 mm), point two millimetres (.2 mm) in width. We measured the crack widths at all stages during the testing.

Now, I just want to show you what to look for because in a moment we're going to have a video of the failure of the south end, but I've just highlighted the crack just to be able to show you what to look for because the failure takes place in a split second and what you should look for is the extension of the shear crack down at the bottom and the extension of this crack horizontally along the level of the number 14's and the number 8 hooks.

So, when we start the video, there's going to be a noise from the testing machine and then you're going to see a white helmet bob up here as somebody walks around and just after that, keep your eyes on this region and this region.

If we could start the video, please. That's the testing machine.

Well, that's what we call a brittle failure. Now, I should point out that it would have been much more dramatic had we not cut the failure at the far end. Oh, I can't show you here. Maybe go back? Oui, *recommence*.

You see it again.

Okay, page 135. Oui.

As I said, this is a very brittle failure and a lot of energy is released and it would have been even more dramatic had we not cut the end of the cantilever, there's a support under here. We didn't want this end to fall totally because we want to strengthen and strap this part of the cantilever, so we can keep loading until we get failure at the other end.

So, I just wanted to mention that it would have just rip right off if we hadn't cut it.

This is the failure, the failure in this region here, we have a shear failure and we have a bond failure along the length of the number 14 bars and this is what it looks like, everything is debonded now and we have that bond splitting failure coming right through at the level of the reinforcement. That's a weak plane where the failure occurred on the as-designed side.

Me MICHEL DÉCARY :

Q- So it runs right along *le plan de faiblesse*?

A- Yes, exactly. Exactly, right along the plane of weakness.

This shows us on the vertical axes the total load applied to the bearing pad at the south and the as-designed end versus the displacement downwards at the location of the neoprene bearing pad, and you can see here is where we were loading up, this is first cracking occurring, we have then the eighteen thousand seven hundred (18,700) cycles of loading at the level which was three thousand and fifty kilo-newtons (350 kN) plus ninety (90) for the live load and we're loading up and down eighteen thousand seven hundred (18,700) times here.

And, at one point, we stopped to replace the joint on the north end which is the as-built end which had a joint replacement in nineteen ninety-two (1992), so we simulated what we thought would be the right number of cycles here, then what you see is us loading the dead load constant on both ends as we replace the joint on the north end, and then, after that, joint was replaced. That's why you see a little bit of displacement there, due to creep deflections, long-term deflections.

And then, we loaded it up, we did a lot of cycling and we eventually got shear failure at eight hundred and ten kilo-newtons (810 kN).

And here is where we strapped the as-designed end so that we can continue loading until we got failure on the as-built end.

Q- So, surprisingly, it's the as-designed that split first?

A- It was somewhat surprising, but we weren't sure because we knew that there were going to be terrible bond problems on that plane and we weren't sure at the time initially whether we hadn't done our non-linear finite element analysis, whether or not we were going to get failure at one end before the other.

And because it was displaced or misplaced reinforcement on the a- built end and there were these horrendous bond problems on the as-designed...

Q- Could you remind us of some...

Monsieur le président?

Me PIERRE MARC JOHNSON :

Q- Which bond problem, I'm sorry?

Me MICHEL DÉCARY :

Q- Yes?

A- The bond of the number 14 bars, remember the number 14 bars didn't have a hook coming down, so that when we have shear, at this section here we have tension in the number 14 bars, due to the bending, you know, you're pushing down here, you're causing tension on the top here.

So, due to the bending, we have tension in the number 14 bars.

But also due to the shear, there's diagonal compression coming and we need tension in the longitudinal reinforcement to resist the shear. So, the shear and the moment cause tension in the number 14 bars and there was a lot of congestion which weakened the bond characteristics of those very large diameter bars.

So, we have a shear failure also affected by the bond stresses in the number 14's.

Q- And in essence, those stresses would have been different had obviously the vertical bars haven't been bonded, in a sense, to the longitudinal bars?
A- It would make a significant difference.
Q- Because they were not bonded, they had this bond problem.
A- That's right.
Q- Yes.
A- That's right. And, if you had that, you still would have had eventually a shear problem, nevertheless.
Q- So, there were two problems...
A- Two problems.
Q- ... as opposed to just one.
A- Absolutely.
Q- Okay.

Me PIERRE MARC JOHNSON :

Maître Décary, je sais qu'il va falloir arrêter quelques secondes...

Me MICHEL DÉCARY :

Oui.

Me PIERRE MARC JOHNSON :

... ou, enfin, quelques minutes même.

Me MICHEL DÉCARY :

Oui.

Me PIERRE MARC JOHNSON :

Est-ce que vous voulez prendre encore quelques minutes pour conclure cette partie-là ou...

Me MICHEL DÉCARY :

Bien, je pense que non, on peut très bien arrêter ici...

Q- Dr. Mitchell?
A- Oui, oui.
Q- Oui, parce qu'on peut reprendre, c'est une autre analyse l'autre côté, c'est-à-dire le tel que construit.

Me PIERRE MARC JOHNSON :

Voilà.

Me MICHEL DÉCARY :

Ça va bien.

Me PIERRE MARC JOHNSON :

Q- You have a nice lab, Doc.
A- Thank you.
Q- You have a nice lab.

Très bien. Alors, nous allons reprendre à et dix (10).

Me MICHEL DÉCARY :

Merci.

Me PIERRE MARC JOHNSON :

À trois heures... à quinze heures dix (15 h 10). Merci.

Me MONIQUE MICHAUD :

Veuillez vous lever, s'il vous plaît.

(SUSPENSION)

Me MONIQUE MICHAUD:

Veuillez vous lever, s'il vous plaît.

Me MICHEL DÉCARY:

Q- So, Dr. Mitchell, we're at page 138 and is it appropriate that we start here at page 139?

A- 139, I'll just switch it, there we go. What I'm going to describe now is what we did to the cantilever and what we called as the as-built end with the as-built details and what we wanted to do was try to replicate, to the best of our knowledge at the time, the replacement of the joint, the expansion joint.

So in order to do that, what we did was we put a constant loading jack on our big distribution beam to create what was the dead load only on the *assise* or beam seat while we carry out the operation of replacing the joint at this end.

And I'll just go through fairly quickly some of the aspects of the joint replacement.

First of all, we gave a saw cut at a prescribed length from where the original joint was, twenty millimetre (20 mm) deep.

Q- May I clarify one thing before we move on, Dr. Mitchell? The joint replacement, am I to understand that you started the testing and you stopped at one point, and then proceeded to replace?

A- That's right. We tried to simulate the likely loading that would have taken place between nineteen seventy (1970) and nineteen ninety-two (1992). Then, we stopped the loading and created a constant load, dead load on the cantilever while we carried out the operations, which did take place in nineteen ninety-two (1992).

Q- And that was done in nineteen ninety-two (1992) with...

A- Yes.

Q- ... as we'll see loads on the structure during the repair process?

A- That's right.

Q- Very well, I'm sorry.

A- Sure. Next, we took the old joint out and we were very careful to chip very carefully in this aspect of the concrete removal and, in retrospect, we were far far too gentle on the concrete if we were trying to simulate exactly what happened on the real structure.

I must point out that we didn't have at our disposal the photographs of the nineteen ninety-two (1992) replacement operation. And if we had those photographs, we wouldn't have been quite as gentle, in fact, we might have been quite brutal in some respects in removing this joint and we also might have removed much more concrete which would have made a very significant effect on the behaviour in my opinion.

So, here we are removing the old joint, removing some concrete. We didn't have the photographs until mid-April and, of course, we were nearing the end of our testing, well past the time when we replaced the joint. So, it was about mid-April that we received those photographs and we were quite shocked at some of the details we saw there.

Me PIERRE MARC JOHNSON:

Q- Does that mean that you didn't impose an additional load on the distal part of the north end?

A- Right. We understood at the time that one half of the bridge was closed during this, while they replaced the joint. In fact, some of the traffic was kept opened on the bridge in one of the lanes and, you know, there were also different types of equipment on the structure which we had no information about.

Here we can see the chipped surface and we see the joint steel Dr. Marchand in his testimony showed the drawings of this steel plate reinforcement detail that was used in the new joint that was put in in nineteen ninety-two (1992). It had number 20 bars passing through holes drilled in the plates, and then number 10 pins that went around like this and another one that went up and around this bar. So, there were a few of those across the width.

Here you can see we've cut off the old steel plate that was attaching the old steel angle detail to the concrete.

And you can see the chipped surface and before concreting, we made sure that the surface was absolutely clean, we made sure there was no standing water on that surface when we replaced the concrete.

And for the concrete, we even did trial batches to be able to get a low slump concrete that was about the characteristics that we wanted for a really good repair on that joint. And that's what it looks like right after the joint repair.

Now, I must say that in reality in many places the joint repair went right down to the bottom. We see in the photographs taken in nineteen ninety-two (1992) that there was some concrete taken out for some distance across the width here as well. And I'll come back to that a little bit later because that gets to be quite severe if you start removing more concrete on the top.

We also took shrinkage measurements because concrete shrinks with time, it gets a little shorter and this has consequences on cracking and the behaviour, especially when you have new concrete placed against old concrete. And here are the shrinkage specimens where we measured the free shrinkage but we also measured the shrinkage on this part of the concrete which is somewhat restrained from the concrete down below the existing concrete.

And from these sorts of measurements, we could determine what the shrinkage strains were versus time and this is the free shrinkage strain and the dashed line is what we call the restrained... shrinkage strain which is somewhat less than the free shrinkage strain.

And now I'm getting into the test itself. This test load was at six hundred and seventy-two kilo-newtons (672 kN) just to show you that the photograph I had shown you previously on the as-designed end was at seven hundred and sixty-two kilo-newtons (672 kN), it's quite a bit higher than this load, yet this end has a lot more cracking due to the joint replacement. We have this horizontal crack which we only had a small horizontal crack at the other end. We see a crack coming in from the corner here and we see some signs of shrinkage cracking along the interface between the new very well placed, very well controlled concrete and the existing concrete.

Also, I must point out as well that our concrete that we were placing the new joint concrete against was not in a deteriorated state. It was really good concrete. And when we're chipping that concrete out, the pieces come out in sharp chunks of concrete, quite a bit different than what we see on the replacement of the actual joint.

The next slide shows you the situation just at failure. We were very lucky, in order to be able to get some detailed readings just as it was failing, we had cameras at each end filming what was happening at all times in case we didn't want to miss the failure but I was able to get right close up just as it was failing to see these cracks opening and take these measurements of these critical cracks that you see here.

And let me just point out what these cracks are. We see some horizontal cracking coming along the bars at this location. Point two millimetre (.2 mm) crack here, a point eight millimetre (.8 mm) crack at this location right by the number 6 hook, the shear crack here incidentally, before failure was point five millimetre (.5 mm). But as I said, this is the rupture load and it was actually starting to fail so I had to stand back so that I wouldn't block the view of the filming that was going on so we could capture the failure.

Q- So, all are equal or most less than point eight millimetre (.8 mm) in size?

A- Yes, absolutely. Right, especially the shear crack, that was a point five millimetre (.5 mm) crack and on the south end it was point two millimetre (.2 mm).

Q- And why stress the importance or you seem to stress the importance of the shear stress, the shear...

A- The crack width.

Q- ... crack.

A- Yes. Well, it is important because when looking at such a structure without stirrups, you need to know the significance of the crack widths and so what constitutes a dangerous looking crack is really what I'm getting at.

Q- So, it doesn't have to do with size only?
A- It doesn't have to do with size only, it also has to do with the length of the crack, where it's located, etc. So, these are things that are really vital when trying to examine a structure and see if it's in a shear distress.
Q- So again I'm repeating it but one last time we'll be coming back to that. So, size is of importance but many other factors also?
A- Yes.
Q- So, one is not to rely on size only...
A- Right.
Q- ... in order to draw any conclusion as to what a behaviour, what exactly is happening?
A- That's right. Now, I've just marked these cracks again to show you what to look for when we show the video of the failure at this end. I should point out that there is this inclined crack coming in here which we can see on the actual structure a little bit later and there is de-bonding that is starting to take place along the interface between the replaced concrete and the existing concrete. So, here is the small crack. Here is the larger crack and here is the shear crack which is going to grow at the bottom here as this grows horizontally.
And if we can run the video please. It's about twenty (20) seconds.

So, if we could return to the presentation. So, I should mention this, I should mention that in this particular case we also caught it before it would reach the floor because we would have to remove all of this afterwards and it's easier if we have some space underneath it. But we let it drop a little bit more than on the south side and you can get an idea of how brittle and sudden that failure was.

So here is the failure at that load level, the next slide shows an elevation view of it. This is what we call the *zone de faiblesse* or weak zone just above the hooks and below the number 14 bars, you can see the classic shear failure going along here are extra number 6 bars sitting right about here and was starting to control this shear crack a little bit and so we didn't get any shear failure there but it tends to be a lot more sensitive at this end here.

Q- You used the term "classic shear crack" why did you use the term "classic"?

A- If you look at a lot of beams that have been tested, they have this S shape, the failure crack has that S shape, failure in shear going up to the compression zone and coming out at typically where normally the support of the beam would be. It's all turned upside down here.

Q- So, this is typical?

A- This is very typical, yes. This shows the detail of the number 10 bars that were...
Q- Doctor, one last thing that just strikes me...
A- Sure.
Q- ... is that you say "typical", would any structural engineer know, recognize what a shear crack is, a typical shear crack?
A- Yes, you know, the shear failure definitely, to judge the significance of a shear crack, it takes a bit of experience though. If it's small, it depends on where it's located, if it's near any reinforcement, if it's in a zone of no reinforcement, how long it is, etc.
Q- But the first element, shear cracking is a well-known phenomenon?
A- Yes.
Q- And structural engineers know about shear cracking?
A- Absolutely.
Q- And they can recognize a shear crack?
A- Yes.
Q- Then what is more difficult as what's a smaller crack or what is the distinction between being able to recognize a shear crack and being able to recognize a shear crack that's more difficult to recognize? I mean, you see there is a matter of degree here?

A- No, what I was referring to that was a bit more difficult was assessing the importance...

Q- I'm sorry.

A- ... of the crack that you're observing and that takes a little bit of experience. Shear crack is a diagonal crack and that is very visible but to judge the significance of a crack, if it's small it becomes a little more tricky.

Q- And as professor is and people of great experience, "a little more"... it's because I'm trying to relate to observations made by Dr. Marchand and it's just a level of knowledge that "little more", to be able to recognize the significance, do you have any other comments you could add on that because to me "little" is -- I'm not being facetious -- is not much more, but really is it a factor of time and experience or is it just the fact of a little more, that is really a bit more?

I mean, you see the degree between an engineer, structural engineer, is it more of a specialist? That's where I'm leading to.

A- Well, somebody that maybe has seen a lot of shear failures, somebody that had some experience in the field, somebody who has observed failures in the lab it really helps when you go to assess a structure. And I'm going to come back to that in some detail a little bit later.

Q- I know. That's why I was just preparing the later examination.
A- Sure.
Q- So, we may go on now to...
A- Now, this photograph here shows the zone of weakness where some of the concrete is even blown out of this particular zone, it was so highly stressed and when it failed in tension and shear along here, it let loose but here you can see the number 10 bars that were added that went and hooked around the number 20 bar passing through the plate that was in the new joint reinforcement but this steel it makes no difference whatsoever in stopping this kind of failure. It just goes around it.

So, these cannot be thought of as bars that project across the plain failure. The failure just skirts right around them. So, those are really not serving much of a function there in that regard.

The next slide shows the load applied at the tip of the cantilever versus the displacement for the as-built side showing the large numbers of cycles that we went through and finally getting rupture at one thousand seventy-five kilo-newtons (1,075 kN). Again, it's very brittle, there is no *plateau* here indicating that something is going to break. It is quite brittle and we were fortunate to be able to see the failure coming with the cracks forming in a rather rapid fashion.

Q- And may we just pause? I understand brittle, in French *fragile*.

A- *Fragile.*

Q- But what does it refer to as applied to the rupture? What is brittle? How would you describe that characteristic?

A- Well, it's something that happens without much warning, with little warning or no warning. It's a sudden failure. There is usually a lot of energy release and that's what would term a "brittle failure".

Me PIERRE MARC JOHNSON:

Allez-y, Monsieur Couture.

Mr. ARMAND COUTURE:

Q- When we look at this slide, Dr. Mitchell, would you say that the displacement at the end, it's a vertical displacement, isn't it?

A- Yes, it is.

Q- Is that related to fatigue in the concrete or is there no relation?

A- Well, we do have the cyclic loadings. So, you can see some effects of the cyclic loading. We were going to the same load level here, for example, in this cyclic region and you see that the deflections are getting a bit bigger and same over here. With cycling we get displacements going from here to there. So, we do see some opening of the cracks. They are very minute openings and we measure those in detail on the structure. So, we saw very small increases in those crack widths with cycling at the same load level, to the same maximum load level.

Q- So, it's related... the displacement at the end is related to the cracking rather than the property of concrete?

A- I wouldn't say the holes but the cracks for sure would dominate the...

Q- Would dominate that.

A- Absolutely.

Q- Okay. On this diagram, would you tell us what would be the design load of dead and live load together against the failure loading?

A- Yes, that's just coming up, I'm just about to get there. I have a slide on that, I think, which is just around the corner here.

Mr. ROGER NICOLET:

Q- Excuse me, just as a follow-up question, are we going to come back to this notion of fatigue or is this the time to open the subject?

A- Sure, I won't be coming back to it, no.

Q- Okay. Because it has been raised in earlier testimony, and maybe it will be appropriate to hear you on your definition of what fatigue is in reinforced concrete as opposed to fatigue as we normally know it on steel members. In other words, are the two phenomena comparable or are they of a completely different origin physically and of complete... are there completely different rules and characteristics?

A- Well, there are some similarities, for example, if you have, if you're looking at the fatigue of say prestressed beam or reinforced concrete beam and you're looking at fatigue in the reinforcing bar that's under tension, it's very similar, then, to the steel case. We have an SN curve which is the fatigue limit curve and what we normally do for a situation like that is we predict the stress in the reinforcement at the crack so that's a bit of a difference between reinforced concrete and steel. So, we would predict the stress and the reinforcement at the crack and we would look at the variation in the stress with the loading, minimum to maximum.

And then, determining the stress variation in the steel at the crack and comparing that to a fatigue related curve for reinforcing bars, and that would be analogist to the steel case, if you like.

Now, if we have a member that doesn't have any shear reinforcement, and we're talking about fatigue and shear, there was a paper written back in the fifties ('50's), I think it was late fifties ('50's), one of the authors was Hessler (Ph.) and he looked at the testing of beams without stirrups and looking at fatigue effects in the shear but they were loading with their fatigue effects up to very very high loads with respect to the ultimate strengths of the beams that were tested and they found an effect, a significant effect.

But those stress ranges were very very high. On a bridge like this, the dead load is quite dominant, both from the hollow box girders and the weight of the cantilever. So, the live loads, yes, have a fatigue effect but I think that the tests that were carried out by the MTQS experts show that there wasn't very much of a fatigue effect because I think they've tested one specimen with cyclic loading and one without so they had a direct comparison and maybe they can comment on that when they give their testimony. But I looked at the results, I made the corrections for difference in concrete strengths and there seemed to be a very very small effect. So, yes, there is an effect, I suspect it's very small in this case.

Me MICHEL DÉCARY:

Q- Well, since it's a subject you won't be coming back, can I ask you first to define what fatigue is and secondly what causes fatigue?

A- Right. Well, fatigue is a failure under repeated loading. If you take a piece of wire and you bend it back and forth, if you bent it enough times it will fail, it didn't fail the first time but it might fail after fifty (50) times bending it back and forth, that would be a fatigue type of failure.

What we see in bridges is because of the truck loading causing increased loads and then decreased loads, increased and decreased, as the trucks pass over the bridge and that causes, say, at a crack in the concrete the steel to stretch a little bit and release, stretch a little bit and release, and that's a fatigue type of loading. And so that's what we normally assess if we're interested in fatigue type loading. We don't do that check at the ultimate state, we do it at what the bridge Code calls the fatigue limit state, which is more like a working load that's to be expected on the bridge.

Q- With Dr. Marchand we, you know, came to appreciate the degradation, the importance of degradation of concrete, how it plays out over time, how concrete ages.

Has fatigue, in our case, played any significant role in the collapse or is it tied more to the causes that are... the three or four causes that are in your report?
A- Well, that's an interesting question, I mean, we're going to see different reasons for the initiation of the crack and one of the effects would be impact on the expansion joint during trucks going over, during snow removal, equipment hitting it. So, that's more of an impact more than it is a fatigue. But fatigue, as I said, this structure isn't very sensitive to fatigue-type loading. Degradation of the concrete is very sensitive to it, especially in that zone of weakness where it actually did deteriorate. So, I put the degradation of the concrete way up there along with the bad details and the misplacement.
Q- And a fatigue would be...
A- Much lower.
Q- ... much lower.
A- It's a factor but I don't think it's a major factor in this case.
Q- So, a fatigue works with time but concrete resists well to fatigue, if I am to understand your testimony?
A- It can.
Q- It can. And in this case did it?

A- I think in this case it probably did. There is a tiny effect.
Q- A tiny effect?
A- Yes.
Q- Thank you. So, we're back to page 155.
A- Right. And this goes back to the question of how does this match with the predictions from the Codes Mr. Couture asked about. And so that's where I'm moving now, is looking at the predictions using the Code approaches. And here, I'm using the two thousand and six (2006) Code to see if we can predict what the failure load would be for the test and the two thousand and six (2006) Code looks at the shear capacity without stirrups, of course, as being composed of two terms, the concrete contribution and the v.c. term, which is the vertical component of the inclined compression on the bottom surface of the cantilever. It's not a large term but I did account for it as well.
V.c. as I've mentioned previously is the resistance due to tension in the cracked concrete. There is tension between the cracks which gives us some shear resistance.
And the two locations where we should at least consider and maybe more if you want to look at all the possibilities but typically we look at the section taken at a distance D from the face of the inclined wall

support. D is the effect of depth. It's the depth, if you like, from the bottom of the cantilever to the centre of the number 14 bars. So, that's D.

So, we come out of distance D from the face of the wall to check the shear resistance.

And also the Code tells us that we should also take a look at a section and calculate the shear resistance, a distance D from the expansion joint. And so that's very important as well, just outside of the disturbed region.

And at this location, the resistance is limited by the capacity of the number 14 bars to develop tension and their ability to develop tension is controlled by the *attirance*, the bond between the number 14 bars and the concrete.

Q- Now, bonding, just if we pause one moment, just to be complete, you noted that there is a bonding by the weight, one stirrup is tied to another, but there is also *l'adhérence*, the bonding, which is the fusion between, in this case it is either l'*acier béton* or *béton-béton*. You want to expand on what exactly is *l'adhérence*?

A- Sure. Well, just to explain it rather simply, if you have a bar and you imbed it in concrete and if you only imbed it a short distance, when you go and you pull on it, it will pull out and it will fail in bond or shear stresses between the bar and the concrete.

So, the question is in our design Codes how far to you have to imbed the steel in the concrete to be able to pull on that bar and develop its full yield capacity. So, that is really what the bond is and our number 14 bars, as I mentioned, go into tension and so we have to look at the bond characteristics of that reinforcement and determine what stress the bars can carry at a particular location.

In this case, we are looking at a location, which is D, from the joint, expansion joint.

Q- Which will be shown on the next...

A- Yes, that is right. So, if we use the two thousand and six (2006) Code, and here, I have shown two possible shear planes and what the predicted shear capacity is according to the Code, at this location here, we have a shear capacity, which would give us eight hundred and thirty-nine (839) kips at the neoprene bearing pads.

So, with a failure plane here, at distance D from the face of the wall, we would predict shear failure when the pad load is eight hundred and thirty-nine kilo-newtons (839 kN), sorry, kilo-newtons. If we come at distance D from the face of the expansion joint, then look at the shear resistance right here, we have a totally different situation than we had over here, we have a smaller moment and we have a smaller shear at this location, because if you are looking at this location, you have this pad reaction times this distance, creating a moment and you have got the weight of the cantilever from this point outwards creating a moment and a shear from the weight and a shear from this load.

At this location, the moment in the shear is smaller and yet, we predict a smaller, I am predicting a smaller failure loaded shear because it can't develop enough stress in the number 14 bars. Shear and moment cause tension in the number 14 bars and the Code tells us, the two thousand and six (2006) Code tells us we have to check the longitudinal reinforcement for the influence of moment and shear.

Q- Which was not the case in nineteen sixty-six (1966)?

A- That is correct, that is correct. So, if you like the predicted load would be eight thirty-five (835), the actual load was eight ten (810) and very close correspondence. This is for the as-designed side. When we go to the as-built side, with the new joint and everything else, we have the misplaced steel and we get a higher load to fail the specimen and, in fact, our predictions are somewhat higher than what we had for the as-built side.

Me PIERRE MARC JOHNSON :

Q- In this case, Professor Mitchell, is it because of these number 6 bars which were not on the plans?

A- The number 6 bars did play a role because the failure crack crosses the number 6 bars and I see that in my non-linear analysis as well, but probably more than that was the fact that these hooks being displaced downwards remove some congestion in the bond at this plane and through some bond parameters, I was able to predict somewhat that effect.

It is a question of estimating the effect because it is quite a tricky one, but because these reinforcing, these number 8 hooks are below the spacing between the bars increases or as if they are on the same plane, you have a smaller spacing of the bars as you have here and more congestion.

So, the predicted load for failure was nine hundred and thirty kilo-newtons (930 kN), the actual one was one thousand seventy-five kilo-newtons (175 kN). So, why we were within three per cent (3%) here on the failure load, we were within thirteen per cent (13%) here.

And actually, in case anybody is wondering, that is very good for a large member like this failing in shear without stirrups to be within thirteen per cent (13%), if you look at the test data in the literature. So, that is strictly from the Code.

Q- And just to put things in perspective, the failure load levels represent a security factor of how much, considering if you could remind us, what the expected load on these?

A- Right, right, the expected load, I do not have in my head because I have looked at so many different...

Q- Yes.

A- ... parameters, but we found that it was, if you look at the design, you use the two thousand and six (2006) Code on the existing bridge, it was deficient in shear by quite a margin in shear. On top of that it is deficient on the hanger reinforcement, this is only the shear, but the hanger reinforcement is perhaps even more deficient.

And so, just looking at the shear is not enough, you also have to look at the hanger reinforcement. It is an inadequate design in accordance with the two thousand and six (2006) Code.

Q- And according to nineteen sixty-six (1966) Code?

A- Nineteen sixty-six (1966) Code, the Code said we did not need any stirrups in this portion and the hanger reinforcement, there were no provisions for the design of the hanger reinforcement.

Q- Thank you.

A- Okay.

Me MICHEL DÉCARY :

Q- Well, maybe just one question because on this part, we see the as built, *tel que construit*, there is two number 6 bars. The Code today says that one of the way of tackling the problem is to insert stirrups. Under the two thousand (2000) Code analysis, where would you have placed stirrups in this part at the *tel que conçu* part that shows no stirrups?

Where would you have placed them, just roughly, without going to detail, not a detailed analysis, but where would you have placed them?

A- Well, first of all, we would have, I am going to try this again. We would have had the hanger steel.

Q- I am sorry, I am not sure I can see you, I am the one that...

A- Can you see it? Can you see it?

Q- Okay, I see it.

A- Okay, so you place the hanger reinforcement hooked around the top bars and this would continue some distance out over the disturbed region, so it is quite a distance out. And then, for the stirrups, these *étriers*, you would have them at a slightly larger spacing, something like this, again hooked around the number 14 bars and the number 8 bottom bars. And that would continue right to the end, something like this.

Me PIERRE MARC JOHNSON :

Q- So, there would have been more numerous, they would have gone approximately towards the angled wall and they would have been positioned differently?

A- Yes, I mean the...

Q- Under the two thousand and six (2006) Code?

A- Yes, well, these number 6 supporting bars were not properly detailed for stirrups, okay, they were only there to support the top reinforcing bars. The stirrups have to go around the number 14's and be hooked around the number 14's and the number 8's on the bottom, so they get hooked around.

Me MICHEL DÉCARY :

Q- Hooked around the *crochet.*

A- Yes, the *crochet.*

Me PIERRE MARC JOHNSON :

Q- Accrochées.

A- Exactly.

Me MICHEL DÉCARY :

Q- Thank you, Dr. Mitchell.

A- Just to conclude on that, the predictions for the shear capacity using the two thousand and six (2006) Code is giving very reasonable predictions for both the as-designed and as-built ends of the cantilever. We feel that the resistance in shear was somewhat increased by the number 6 bars, but they may be not that effective because they are not properly anchored.

And if I just move on to the next slide here, just to come back to the earlier slide that I showed, where we were looking at the size effect in the shear, just to put on this spot, for the case that we have with an effective depth of about four feet (4'), remember that the original design under working load, service load, was fifty eight (58) PSI, the nineteen sixty-six (1966) Code limited the shear stress under working loads to seventy (70) PSI and the test failed at a shear stress of seventy-nine (79) PSI.

So, that failure shear stress is fairly close to the test carried out in Japan. Now, we are ready for another part of the presentation, the non-linear analysis of the cantilever and just before I start, non-linear analysis once again is breaking the structure that you are looking at up into small elements and linking them together appropriately and looking at equilibrium and compatibility between the elements to make sure that you can predict the stresses.

And when you go non-linear, you are then able to account for the presence of cracks, you are able to account for the yielding of reinforcement, you are able to account for bond stresses if you use a fairly sophisticated program.

So, what we have done is done some analysis of the slices, we have used a commercial program called Vector developed by Dr. Vecchio at University of Toronto, it is a very sophisticated program for non-linear analysis and we used the post-processor called Augustus that was developed by doctor Benz (Ph.), also at the University of Toronto.

And what you see here is the model that we used to predict the response of the as-designed end of the cantilever. You can see the *suspente numéro 8* in green with the hooks on them at almost the same levels than number 14's, the number 6 inclined reinforcing bar, the U bars in pink, the number 14 bars in red and the zone we see over here, the funny shade of blue, is where we have got the inclined reinforcement coming out of the wall.

And these are the pad loads that we have over here and what we do in the non-linear analysis is we start from zero load and we increase the load in increments until we predict failure of the element we are looking at.

So, there is almost two thousand (2000) elements in this model and this is a prediction for the as-designed side at failure and I just wanted to show you a few things, it is very distorted because the failure has come right through this zone here and skirting around and down in a classic shear mode.

You see the flexural cracks fairly widely spaced and you see the cracking near the region where the number 6 inclined reinforcement is in the first leg of the number 8 bars, slight over prediction, but within reason, considering that we are looking at all of the different complex features going on in this element, we also looked at the non-linear analysis – so we just have a minute here – what we are going to look at in a moment is the modelling using the program Vector at the as-built end where we had to use different concretes, particularly where we have the joint replacement because that concrete has different properties than the concrete in the rest of the cantilever.

We have actually stronger concrete, stiffer concrete and we have concrete that's shrinking relative to the existing concrete in the cantilever, and so, we wanted to model that appropriately and also include the number 6 bars that were there providing some sort of function in resisting the loads.

Q- And I think we should come up to the video?
A- Right after, yes, right.
Madame Desprez, il y a certaines langues ici qui pensent que c'est en raison d'un geste de ma part que...
A- We have a blackout here.
Q- Voilà.
A- Great. So, there is what I was describing and I have lost my controls here. Okay, there is the video and, in fact, I can initiate it, I believe I can initiate it from here. Well, what you should look for is that as the load is increased on the end, this is our modelling with non-linear finite elements, you will see – here we go – you will see some cracking on the right flexural cracking, then you will eventually see the critical shear crack developing and there it is, so that is the failure right there. So, this is our model of how the cantilever would deform and the red lines are showing where the potential cracking is and there was a little bit of... quite a bit of distress around the joint area as well.
And if we compare that prediction, and I will try to get my pointer working back here.

Me PIERRE MARC JOHNSON :

Q- I understand that the analysis, the predictive results came out before the laboratory testing results?

A- Yes, well, we had made a prediction and what we did is we had to change the shrinkage on the new concrete and the joint area, because we were not sure how much the shrinkage, so we had to do an analysis on that, but basically, yes, right. And what you see in this figure here above is the predicted failure plane and below you see the failure plane in the actual test and they resemble each other very closely.

Mr. ROGER NICOLET :

Q- At what load level?

A- This is the one where we were predicting, this load level was nine hundred and fifty kilo-newtons (950 kN) and that is the prediction, the test was one thousand seventy-five (175).

Q- Thank you.

A- Yes. I cannot control it from here, I cannot go to the next slide. Okay, merci. So, we get the same sort of prediction, we get a brittle failure in shear and we have sufficient precision with the prediction to look at the mode of failure and the progression of the cracking.

The different factors that came into play during the life of the structure, and really here, I am going to be talking mainly about the replacement of the joint and some of the cracking that we have observed at the evidence sight. Le prochain.

This is a view of one of the slices near the north-east corner of the cantilever and this is on the other side of the bridge across the lanes from where the failure took place. But it is clear, I cannot point, but there is severe degradation along the crack, the crack is following right along the top of the number 8 hooks, I hope you can visualize that and the number 6 diagonal bars, the hooks of the number 6 diagonal bars.

Me MICHEL DÉCARY :

Q- May I interrupt, Dr. Mitchell, for a moment?

A- Sure.

Q- Madame Desprez, est-ce que nous sommes en attente du marqueur, deux minutes? Je pense que ça vaudrait la peine d'attendre.

A- Oui, oui.

Q- If you do not mind, Dr. Mitchell.

A- Oui, oui.

Q- It will not be long, Messieurs les commissaires, parce que la démonstration qui suit est importante et si vous me permettez, si c'est plus que deux minutes, évidemment, on poursuivra.

M. ARMAND COUTURE :

En attendant, vous permettez que je pose une question au docteur Mitchell?

Me MICHEL DÉCARY :

Oui, oui, oui.

Mr. ARMAND COUTURE :

Q- In the non-linear simulation, can you put the degradation of the concrete as one element of the simulation?

A- Absolutely and that is coming up. Yes, we actually tried to, because I have been able to do that before on some other structures, you know, showing degraded concrete and we put it into the model and we tried to simulate stage-wise degradation of the concrete. Yes, it is coming up.

Okay, we are back in, we are back in business here. So, this is the degraded concrete that you see here, you will notice the degraded concrete on the top surface as well, it is quite significant. This is a very severe crack coming along the length of the hooks as you see and the crack comes out through the top here somewhere and we think that water was able to get in through this and feed into this slightly inclined surface that has opened up with the crack.

We have the large cracks, we have the degraded concrete and when Dr. Marchand and I first visited the site, and we were looking at block CNE1 before it was sliced up, and we walked up to this region and this part of the concrete was so soft, you could take your fingernail and press it on the concrete and it would sink into the concrete.

I mean we are talking about severely degraded concrete just above the hooks on this specimen, and there is one other item I should perhaps comment on at this time, because during the cross-examination of Dr. Marchand, it was suggested that perhaps the cracks have become larger since these specimens were transported to the Belgrand site, but I would like to point something out here because remember that this cantilever was fixed away over on the right-hand side and was sticking out some thirteen feet (13') on the real structure.

And there were significant loads coming down on the tip of this cantilever from the weight of the box (inaudible) bridges is ninety foot (90') spans and that was loading the *assise* or the beam seat here with quite a substantial load. And in addition to that, you had the weight of the cantilever itself with this crack opened and my point is that, in this mantling, taking the beams off, the crack probably got smaller.

And taking this piece over to the evidence site and sitting it down on supports here whereas it used to be sticking out as a cantilever probably make the cracks smaller, because you have got a direct support underneath this part, it was originally hanging below the cracks.

So, I do not think it is right to say that the cracks are probably larger because of the transportation. I would contend it is the opposite, the cracks were probably smaller than what they were on the actual structure.

Me PIERRE MARC JOHNSON :

Q- On that, Dr. Mitchell, how many inches or feet are we inside the north-east structure here?

A- That is, see, I think each of these is about two or three feet (2-3'), I would estimate here maybe about six feet (6') in, something like that.

Q- Six feet (6') in?

A- Yes. You are away from the sidewalk, which was on the outside here. So, you are probably about six, seven feet (7') in on this face, my guess. Now, Dr. Marchand talked about the degradation of the concrete and what we have now termed the *zone de faiblesse* and if you take a look at this photograph here, especially the close up, this is the failure plane right here.

So, we are looking at the concrete immediately below the failure plane and it is all layered, it is in slivers, I mean the concrete has no tensile resistance in this direction. It was just pulled apart and you saw how we had to carefully monitor that piece of concrete that we had in the room the other day.

In addition, if there is to be any shear, I am trying to get the... if there is to be any shear stress across here, this would just slip, there is no shear resistance any more along that plane. So, this is very serious concrete degradation that we are talking about in this *zone de faiblesse*.

And Dr. Marchand also showed that there was considerable deterioration for some distance above the *zone de faiblesse* on the other piece at the site, the evidence site. Now, this is a photograph of the sister bridge to the Concorde bridge, the de Blois bridge, just immediately north of the Concorde bridge, or at least was.

And this is a photograph in nineteen ninety-two (1992) before the joints were to be replaced on the de

Blois bridge and the Concorde bridge and we just wanted to point out here that these expansion joints take a terrible beating, not only from truck traffic, but also from snow removal equipment, hitting the steel angles.

You can see here the steel angle has been torn right out, there is great big holes in the joint area for water to get into, there is standing water over here, it is not draining properly. So, if you look at what this might have done, this might have been one of the initiating causes to the cracking that occurred in the *zone de faiblesse* or the weak zone.

This next slide shows some heavy equipment on the bridge during the removal of the asphalt and we realize that this probably was not there, we say "probably was not there", during the chipping of the concrete to replace the joint, but this is a Caterpillar Cat 235, which has a weight listed in the catalogue of ninety-two thousand pounds (92,000 lbs.).

The weight of an H20-S16 truck for which the bridge was originally designed is seventy-two thousand pounds (72,000 lbs.). I have to point out also that the Cat 235 is shorter than the twenty-eight-foot (28') long H20-S16 standard truck.

However, one thing in its favour is that the distance between the treads is eight feet (8'), it is wider than the standard truck, which is only six feet (6') wide. So, it spreads it out more laterally, but less in the longitudinal direction.

Mr. ROGER NICOLET :

Q- What would you say about the ability of the bridge deck to distribute the load laterally?

A- Well, that is a good question, there are fairly wide treads we have... we have a topping on top of the hollow box bridge girder, so it's a three and a half inch (3 1/2") topping, I think it is probably, probably okay, but what we did is we did an analysis using our three-dimensional model of the entire bridge and we found that if you're looking at the sheers in the cantilever, which is a vital part of the design here, that the shears in the cantilever with this leading would give a shear per unit width which is twenty-five per cent (25%) higher that a standard H20-S16 truck, twenty-five per cent (25%) higher.

However, the bridge was designed for two lanes loaded as well for the H20-S16 trucks and if you compare the shear, the designed shear, that you would have back in nineteen sixty-six (1966) with the shear from the Caterpillar C Cat 235, we found that it gives a five per cent (5%) lower shear than the two-lanes loaded with H20-

S16, but still a considerable load.

And it might have been - and I say might have been - one of the largest loads the bridge had ever seen in its lifetime.

So, that's important for...

Me MICHEL DÉCARY:

Q- And, Doctor, you just said "it might have been the largest load", what do you base yourself upon to make that statement?

A- Well, we had a transportation study which gave us the probable range of vehicle weights and of course those are different vehicles altogether, but they wouldn't have been as heavy, they wouldn't have created this higher shear as this here. And the other thing I'd like to point out is...

Q- Yes.

A- ... that not only is there a Cat 235 on the bridge but there are other vehicles as well. So, I mean, we have to recognize that this was a very serious load test on the bridge.

Mr. ROGER NICOLET :

Q- Coming back, I'm sorry, to this...

A- Sure.

Q- ... lateral distribution across the bridge deck, there are no, at the support, there are no cross girders between the box beams and, if you look at the behaviour of the cross beams further towards the mid span, it seems that the prestressing wasn't particularly high, which resulted in some longitudinal cracking in the bridge deck. Did you notice that or did you make any... take any conclusions from that observation?

A- Yes, the diaphragms, there were no diaphragms at the end, the diaphragms in this band were... had post-tensioning, transverse post-tensioning going through them, as far as the cracking on the deck is concerned, that was in the asphalt, I believe that what we had reported, had reported to us that there was some cracking in the asphalt, not altogether indicative of cracking in the member below, though.

So, I didn't want to make the assumption that that cracking had existed, but it's a good point, this is a very heavy load on the individual box girders, the top flange with the topping.

There is one other item I'd like to point out on this photograph and Dr. Marchand showed on Friday, I think it was Friday, yes, Friday during cross-examination, one of the number 14 bars that was severely bent up during the removal of the joint and what we see here is due to the *pelle mécanique* or the backhoe that you see here, a bar that's been ripped out of the concrete. You can see that in the foreground here.

And there is another photograph of that bar from a slightly different angle and it's been quite destructive in removing some of the asphalt there, obviously some concrete was removed as well.

Me MICHEL DÉCARY :

Q- What would that suggest, Dr. Mitchell?
A- Well, that would suggest that the backhoe, the shovel did some damages and perhaps it wasn't intended, and obviously this bar is ineffective now, and there may be other bars as well.
Q- It says something about the way the repair was conducted?
A- Yes. Yes.
Q- That is not in a gentle way?
A- It wasn't in a gentle way as we had conducted it in the laboratory, that's for sure.
Q- It was maybe the use, a certain expression that I use, in a semi-destructive way?
A- I don't want to go there.
Q- Don't comment. Don't comment.
A- I don't want to go there.
Q- That was a low blow, I recognize that.

Me PIERRE MARC JOHNSON :

Q- Would you have stood besides that Caterpillar?

A- If I had seen the structural drawings ahead of time? No.

Q- So, if you had seen the structural drawings...

A- Right.

Q- ... you would not have stood...

A- Right.

Q- ... on that Caterpillar.

A- Right.

Q- Not that you're that heavy, but...

A- No.

Q- Okay. Thank you.

A- Okay. This is the replacement of the joint during nineteen ninety-two (1992), the digging out of the concrete and I hope, clearly, it's not as evident maybe for the audience, but the hooks of the number 14 bars are definitely below... sorry, the hooks of the number 8 bars are definitely below the number 14 bars, and Mr. Sonago who took these photographs or had someone take these photographs, testified that he noticed this fact. He noticed that the hooks of the number 8's were below the number 14 bars, and he claims that he requested additional reinforcement to be placed in here, we don't have any details of what he wished to put in there, but he was talking about twenty millimetre (20 mm) diameter bars and these have become a bit of a mystery because no one has seen those bars, and obviously they were never placed, we don't have any idea what they were going to look like and so we never found these bars, even during the dissection of the elements.

Me MICHEL DÉCARY :
Q- But, Dr. Mitchell, had you seen this, what appears on page 173 in nineteen ninety-two (1992), on top *il y a la fenêtre d'observation*, you see... I mean there's an opening now, you see all the structure, what would you have done?
A- Well, I would have closed the bridge. I would have closed the bridge, that's right above a hanging support, those number 8's are in tension, are meant to be in tension. I would have closed the bridge, and...

Me PIERRE MARC JOHNSON :
Q- Because of the depth of the extraction of concrete...
A- No.
Q- ... or because you notice there that the vertical bars, which then become horizontal, are not where they should be, they're lower than they should be, or both?
A- Well, what concerns me the most is they're not hooked around the number 14's, I mean, yes they're lower and

that's another feature, but this is a hanging support and those bars have to be hooked around the main flexural steel, which are the number 14 bars.

So, I would have closed the bridge, supported the bridge and then a study would have had to be carried out to see what's going on, I mean, this is a very bad detail.

Me MICHEL DÉCARY :

Q- Well, just to start with the premise, you would have seen this, the first thing that struck you is that they had to be hooked?

A- Right.

Q- Well, was that well-known in nineteen ninety-two (1992)?

A- Well, that's a good point, I suppose somebody on the site would have the structural drawings, at least one on site could have caught that it was misplaced. At least that. And then, a call should have gone in : what do we do about it? I mean this, in nineteen eighty-four (1984), we had proper details and proper design procedures in CSA A23.3, which is the Code for the design of concrete structures, not the bridge Code, but we had those in place then.

So, designers were, you know, since nineteen eighty-four (1984), so that's eight years later, were well familiar with disturbed regions and how you should design them, hadn't reached the bridge Code, but at least someone should have flagged the misplacement and something should have been done.

Me PIERRE MARC JOHNSON :

Q- So you're saying there's the nineteen sixty-six (1966) Code which was used to do the design...
A- Right.
Q- ... which gave way to a detail of the upper part of the structure which was a detail which proved to be inadequate, according to two thousand six (2006) Code.
A- Yes.
Q- But, what you're saying is that there was nineteen eighty-four (1984) Code modifying the nineteen sixty-six (1966) Code.
A- No, the eighty-four ('84) Code was for buildings, if you like...
Q- Okay.
A- ... for building structures rather than bridges, but that Code gave... and the follow-up design examples, which I presented earlier, gave detailed examples of joints like this.
Q- Would you say that the normal structural engineer, average structural engineer, would have known that in

nineteen ninety-two (1992)?
A- I think so, someone who is keeping a breast of...
Q- Of developments.
A- ... of his profession, yes. Yes.
Q- Thank you.

Me MICHEL DÉCARY :
Q- And if not, if not that knowledge of, you know, awareness of the advancements of science, at least the first trigger would be the fact that the actual building... structure, I'm sorry, did not conform to the plan...
A- Absolutely.
Q- ... is it normally not a trigger?
A- Absolutely. Absolutely. It should have been.
Q- And what do you do, once you realize there's a mismatch, what do you normally do? I mean, you call someone in Quebec city or somewhere, what, I mean, someone has to give a... what do you do once you realize that there's a mismatch? What's the next step?
A- Well, that's right, get the design team in and... first, first close the bridge; second, support the bridge; then call and get the design team in, find out what is going on.
Q- And maybe just to pursue the matter, find out what is going on, how do you go about finding out? The bridge is

closed, you added support, can you just be more explicit on adding support, and then the analysis or... I use the term "analysis", but I mean what are the next steps?

A- Well, you get the plans out, you look for all signs of distress and deterioration on the structure, including cracks. There were some signs of cracking and putting two and two together, no hooks around the number 14, misplaced steel, cracks, I think you start to see the entire picture, which I'll show a little bit later.

Q- So, I'll let you... I will come back to the answer to that question as you move along.

A- Okay.

Q- So, just to conclude on that - I'm sorry to... Monsieur le président, but I mean you would have closed the bridge having seen the mismatch between the bars and the plan and also specially because of the fact that they're not tied or bonded or hooked to the longitudinal bars?

A- That's correct.

Q- Fine.

Mr. ARMAND COUTURE :

Q- This...

Me MICHEL DÉCARY :

I'm sorry, there's was a question. Monsieur Couture..

Q- Dr. Mitchell.

Mr. ARMAND COUTURE :

Q- Dr. Mitchell, when you look at this picture...
A- Yes.
Q- ... can we conclude that some of the asphalt on the deck was removed only after the joint was open? Is it the asphalt we see on the right side of this picture?
A- I think that's the asphalt on the right, right here, yes, I think so.
Q- So, we know it was removed, so it was removed after the joint was open?
A- I'm not sure about that. I can't say for sure.

Me MICHEL DÉCARY :

Monsieur le président, il y a une analyse ici et qui est assez longue et vu qu'il est presque et demie, je vous demanderais de pouvoir interrompre le témoignage du docteur Mitchell à ce stade parce qu'il y a une suite et qui ne peut pas être faite en deux minutes ici, et c'est une suite qui est préparatoire à des questions, à certaines questions d'importance. Donc, je vous demanderais, s'il est possible d'ajourner à demain matin à ce stade, à moins que vous décidiez de continuer.

Me PIERRE MARC JOHNSON:

Avant d'accueillir votre requête avec le sourire, Maître Décary, vous allez nous expliquer un peu où on s'en va avec le docteur Mitchell dans les vingt-quatre (24) prochaines heures peut-être.

Me MICHEL DÉCARY:

Well, Dr. Mitchell...

Me PIERRE MARC JOHNSON:

Vous avez remarqué que je n'ai pas dit quarante-huit (48).

Me MICHEL DÉCARY:

Q- See, as a lawyer, you know, I don't want to answer that question but I'll ask someone who not only has an engineering degree, a masters degree, a doctors degree, un diplômé de l'école de génie, Dr. Mitchell, your estimate in time, how much time?
A- Oh boy!

Me PIERRE MARC JOHNSON:

Q- Don't be dragged into this.

A- Well, we could get to a natural stopping place in three more slides. But then I'm not anywhere near the end.

Me MICHEL DÉCARY:

Q- No, no, but how much time would you have to get to the end?

A- For the whole thing?

Q- Yes, for the whole thing.

A- For the full thing, just let me make an estimate here. About an hour and a half.

Q- Deux heures, Monsieur le président.

Me PIERRE MARC JOHNSON:

Okay, so maybe we can go through the next two slides.

Me MICHEL DÉCARY:

Yes.

Me PIERRE MARC JOHNSON:

Q- And I understand that the video that we wanted when you presented your analysis is the one we got. There was no other video to be seen.

A- Yes, that's correct, that's correct.

Q- Okay. So, let go through to the next two slides, and then I'll have to consider maitre Décary's request.
A- I just want to warn that after I talk about this next one, I need COM-1-C, page 87. So, looking at the damage on the reinforcing bars, Dr. Marchand talked about this on Friday and these are pits in the steel bars due to hits from jack-hammering on the bars and we know that if you do this, you not only have a chance of reducing the area, effective area of the steel, reinforcement at the section where it's hit, but also when you hit the bar heavily you tend to lose some bond of the bar. I'm not saying that that's what you see here but if you hit the bar, it vibrates and shakes it relative to the concrete and that can damage the bond between the steel and the concrete.

And I just want to bring up COM-1-C, page 87. Now, this is the photograph that Dr. Marchand also showed about the removal of the joint using a hydraulic hammer and I can't point but if we could zoom in to right around where the...

Me MICHEL DÉCARY:
Q- Madame Desprez, oui.
A- Parfait. Now, this is a hydraulic jackhammer and using it on the joint and the reason we got some certainty

Audio Sténo Transcript, sténographes officiels, 510 Boul. St-Laurent, suite 200, MTL, QC, H2Y 2Y9

about that is that this is the joint reinforcement that has being dragged out of it. You can see the... sorry, you don't know what I'm point at but just to the right of the gentleman with the red and black jacket on, there we go, that's the steel angle from the joint and you can see the steel plates sticking out of the steel angle which are welded to the steel angle, there we go, so that's the joint reinforcement that was removed with this type of equipment and that's very heavy equipment and does damage to the bars and may do damage to the concrete as well.

The other feature that I wanted to bring out, and this is on the south-east half of the bridge, fairly close to the... this is in the outer lane, not in the inside lane, sorry it's in the inside lane, not the outer lane, the exterior lane, but what we see here is the new expansion joint reinforcement with the number 10 hooks. This is typically about four hundred and fifty millimetres (450 mm) and when we did our repair in the lab at McGill, that's the amount of concrete we removed.

But in this case, there is being an enormous amount of concrete removed and the reason I say it's significant is that this concrete removed even below the number 14 bars reaches the second hook of the number 8 hanger reinforcing bars. So not only has the concrete being removed above the first leg of the *suspentes* or number 8

U bars but also the second leg in some places. This is very serious, very severe. We didn't do that in our test in McGill because we only got the photographs of the nineteen ninety-two (1992) joint replacement in mid-April.

Q- Could you expand as to why it's very serious?

A- It's very serious because it's removed the concrete and the connection between... the only connection that was remaining between the second leg and the top number 14 bar, tension in the concrete.

Q- Yes.

A- And later when you place new concrete over here, you're going to have a plane of weakness if the joint wasn't properly concreted where you could get cracking along the interface of the new concrete with the old and that is starting to look like the failure crack that occurred. It's along that plane. So, that's why it's of great concern.

Mr. ROGER NICOLET:

Q- To better understand this picture, are we seeing cross-wise the old width of the bridge? And in the back there, this is the mid-strip?

A- This is the median here.

Q- Yes.

A- And that's the expansion joint going through the median. This is the lane beside the median and this is the... if you go towards this end here, you're going towards the south-east corner.
Q- Okay.
A- So, this is close to the region.
Q- But in other words, the width... we understand from Mr. Sanogo's testimony that what he removed was the concrete that he thought was damaged. Would that explain why the dark back... as far back as you show here on this picture?
A- Yes, absolutely. I mean, they have to remove damaged concrete. It's just that it happens to have been done in the most critical of locations...
Q- Yes.
A- ... where the hooks are missing from the number 8's.
Q- But the failure was triggered closer to the south end of...
A- Right.
Q- And not in the area where...
A- That's right. It's towards the front of the bottom of this picture but we don't know how much concrete was removed there. So, this also would cause distress, I mean, we see that there were shear cracks... the plane of weakness cracks were apparent on the north-east corner as

well, so that this certainly does not help the situation. It makes it worst.

So this defect, first of all, should have been reported immediately and something done about it. And yes, it's a catch 22, you're supposed to remove deficient concrete, degraded concrete but above the hooks makes it particulary dangerous actually.

Me MICHEL DÉCARY:

Monsieur le président, avant l'ajournement, on m'a indiqué qu'un certain nombre de chercheurs à travers, enfin j'oserais dire, à travers le monde, des chercheurs dans les universités, les écoles de génie de l'industrie du béton s'intéressent aux audiences, aux travaux sûrement mais aux audiences, aux témoignages et ils nous ont fait part d'une demande s'il leur était possible d'avoir une disquette sur laquelle on retrouverait la déposition intégrale des experts. L'on sait que ce que l'on entend à l'heure actuelle est la traduction et des questions et des réponses et il semble qu'on n'ait pas accès à la version intégrale et on nous a fait part qu'évidemment d'abord, c'était en français, les difficultés que ça peut poser à l'étranger, et enfin s'il était possible d'avoir la version intégrale en français.

Me PIERRE MARC JOHNSON:

En anglais.

Me MICHEL DÉCARY:

En anglais, en anglais, merci. Pardon.

Me PIERRE MARC JOHNSON:

Alors, j'ai été saisi de cette question tôt ce matin. J'ai demandé aux gens qui s'occupent du système de diffusion de trouver une des deux solutions suivantes et je vais le dire en anglais, considering the people who have an interest in the expertise of our witnesses and would like to have their testimonies in English rather than have their voice covered up by a translation, and I have asked the people responsible for that at the Commission to see to it that in the coming days one of two things is available, either on the web we will create a system by which the testimonies can be heard in their original version and the translation is on a second track or if that's not possible, we will have indeed either available through the web or on cdroms the original version, and then people will need to contact the Commission to get them.

We should have a clear answer for that tomorrow as we open or close. If you're doing this right, maybe as we open and if you're too slow it might be at the end of the day but we'll have an answer tomorrow on these issues.

We will consider a cdrom which includes, of course, the pictures and not only the audio.

Me MICHEL DÉCARY:

Which could be made available upon request to the secretary.

Me PIERRE MARC JOHNSON:

Which would be made available upon request. So, tomorrow we'll have details about that and of course we'll post the details on the web site.

M. ROGER NICOLET:

Does that include the testimony of the experts for the other parties in this section?

Me MICHEL DÉCARY:

In all likelihood, yes.

Me PIERRE MARC JOHNSON:

I would take for granted that to make all these expertises available, we'll make them available from all the experts in the original language once again and with available separate translation.

Alors, Maître Décary, vous suggérez qu'on recommence à sept heures trente (7 h 30) demain matin? Non?

Me MICHEL DÉCARY:

Je m'adresse à vous pour la décision, Monsieur le président, puis je n'ai pas de demande à cet effet.

Me PIERRE MARC JOHNSON:

Alors donc, croyez-vous qu'on est prêt à neuf heures trente (9 h 30) demain?

Me MICHEL DÉCARY:

Oui, neuf heures trente (9 h 30) sûrement, oui.

Me PIERRE MARC JOHNSON:

Oui.

Me MICHEL DÉCARY:

Oui, sûrement. Et on a fini certainement demain matin, mais évidemment, là, mes confrères ont des contre-interrogatoires et peut-être une note, si je peux me permettre, m'adresser d'abord à maître Arguin, vous prévoyez un contre-interrogatoire d'à peu près combien de temps?

Me PIERRE ARGUIN :

Vous voulez me mettre sur la sellette. Je ne crois pas que ça sera un contre-interrogatoire de quatre heures. Ça sera certainement moins que ça mais écoutez, je prévois peut-être quelque chose entre une heure et deux heures, peut-être un peu moins.

Me MICHEL DÉCARY:

O.K.

Me PIERRE MARC JOHNSON:

Maître Henry.

Me PATRICK HENRY :

Afin que je puisse vous montrer que je peux respecter ma parole, je vais dire une demi-heure maximum.

Me MICHEL DÉCARY:

Donc, ça sera très court.

Me PIERRE MARC JOHNSON:

Donc, en principe, on pourrait en terminer avec le témoignage du docteur Mitchell demain si tout le monde respecte ses engagements et peut-être même amorcer un prochain témoignage avant la fin de l'après-midi.

Me MICHEL DÉCARY:

Oui, sûrement, oui, docteur Bissonnette est prêt. Et peut-être une dernière note, si vous aviez, Maître Arguin et Maître Henry, des demandes spéciales à adresser à madame Desprez, c'est-à-dire des pièces, vous pourrez leur donner sans qu'on en soit informé mais ce soir pour que ça puisse être prêt... pour qu'elle puisse être prête demain matin.

Me PIERRE ARGUIN :

On pourrait le faire par courriel, j'imagine? Oui ? Bon. Alors, je verrai.

Me MICHEL DÉCARY:

Très bien. Merci, Monsieur le président.

Me PIERRE MARC JOHNSON:

Merci. Alors, nous suspendons nos travaux jusqu'à demain neuf heures trente (9 h 30).

Je, soussigné, ROBERT TÉTRAULT, sténographe officiel dûment assermenté, en français et en anglais certifie sous mon serment d'office que les pages qui précèdent sont et contiennent la transcription exacte et fidèle, au meilleur de mes connaissances et de mon jugement, de l'enregistrement mécanique effectué hors de mon contrôle.

Et j'ai signé,

Robert Tétrault

ROBERT TÉTRAULT, s.o./o.c.r.