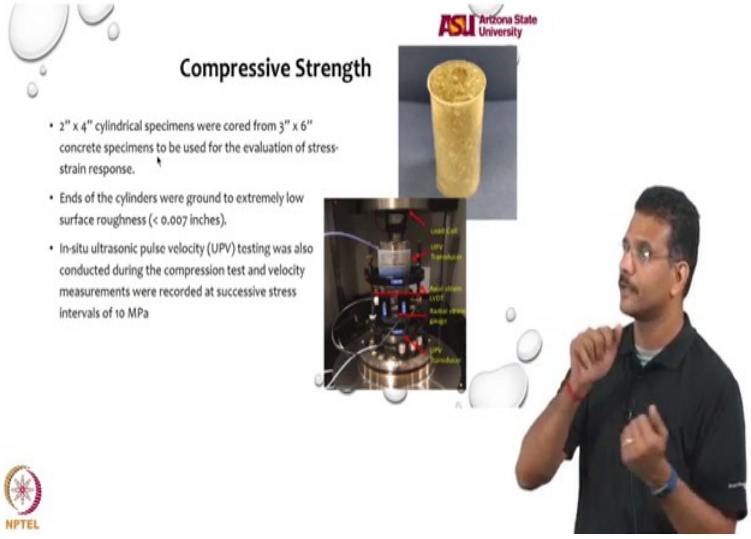
Advanced Topics in the Science and Technology of Concrete
Professor Narayanan Neithalath
School of Sustainable Engineering and the Build Environment
Arizona State University, Tempe, AZ
Ultra High Performance Concrete (UHPC): Material Design and Properties
Part 2

(Refer Slide Time: 0:12)

The next part of the talk will be on the properties of ultra high performance concrete and I will show you only a few properties of interest.

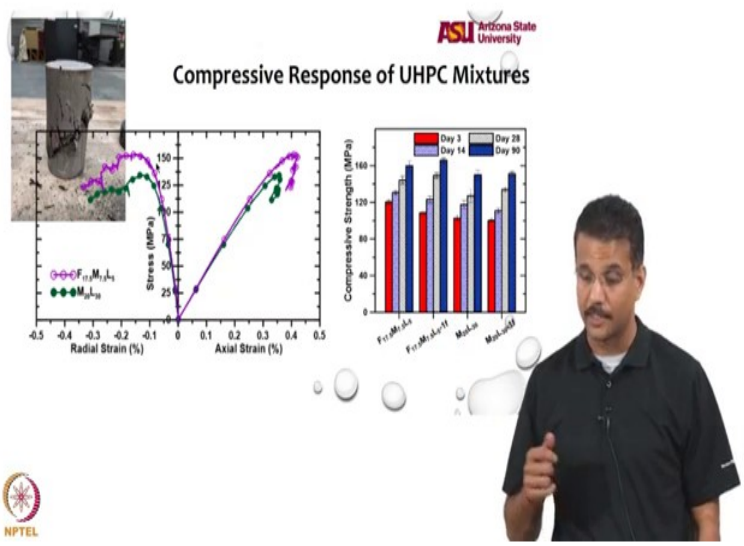
(Refer Slide Time: 0:28)



So let us talk about compressive strength. Now, if you just do a simple compressive strength test, you will say that I have 150 or 160 Megapascal strength. But, like I showed you earlier, the reason why we use ultrahigh performance concrete is not just for strength, strength is one thing but we also want to know how we can use this material in designing structures. So if you do the design of reinforced concrete structures like bridges or girders or columns, you actually want to know the complete stress strain response and the lateral response.

So now the testing has to account for both the axial deformation and the lateral changes in dimension, so you have to do both axial and lateral. If you have to do that, and we are doing this in displacement control mould in a closed-loop testing with a radial strain control because your radial strain is much more, you have overall volumetric strain is axial strain plus twice the radial strain. So your radial strain is very sensitive and therefore you have to control the radial strain after you have loaded to about 50% of the peak. There are couple of different load changes that you have to do in this kind of testing and we did ultrasonic pulse velocity testing as the specimen is being loaded so that we can do non-destructively the elastic modus under different loading stages.

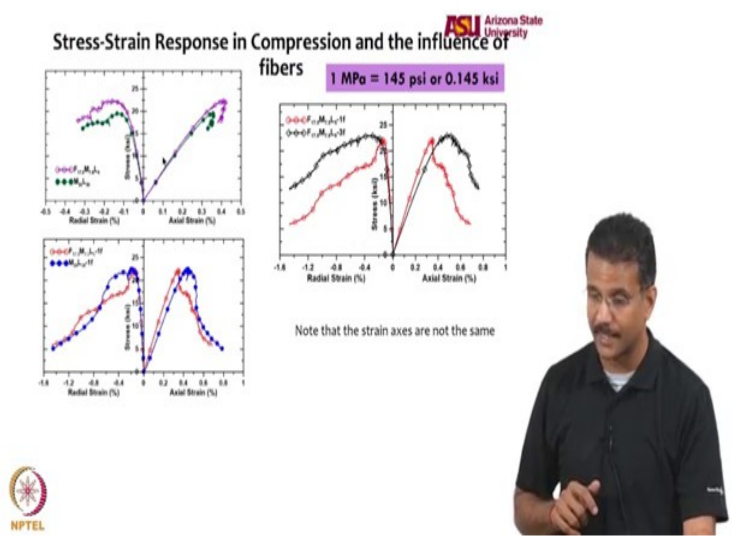
(Refer Slide Time: 2:08)



So here is the complete axial stress axial strain and axial stress radial strain curves for non-fiber reinforced UHPCs. You see the maximum strength is 150 Megapascal, the maximum axial strain is of about 0.3, 0.35%, which is basically the maximum allowable strain of concrete in the sign. So first understanding is that the ultrahigh performance concrete does not change the maximum axial compressive strain capacity of concrete in unreinforced state.

The strength as we can see is up to 160 and 170 Megapascal depending on the kind of mix, and you will see the failure pattern of a mix right there, again very high strength mixes fail in this fashion as you would have seen.

(Refer Slide Time: 3:09)



Now, when you look at the influence of fibres, this is a little tricky figure because the stress axis is in kips per square inch which is 1000 psi. So the left top figure is the same one that I showed earlier, which is basically the axial strain and radial strains as a function of axial stress for unreinforced mixes and the one on the left bottom is for a mix with 1% fiber.

Also, notice that the axial and radial strain axis are different. The first one goes only to 0.5, the second one goes to 1 for axial strain and -1.6 for radial strain, the axial strain is increased because of the effect of fibers post first crack. You will also notice that the maximum axial strain at the peak has not changed, it is still around 0.35, which means from a design point of view that's what you should use because in reinforced concrete design you look at what is the strain at peak load.

Now, if you look at the left bottom picture, the radial strain has significantly increased with increasing fibres because if you push in compression, because of fibres confining the movement you can have much more radial strain without failure, because fibers are holding and bridging the cracks and all of that.

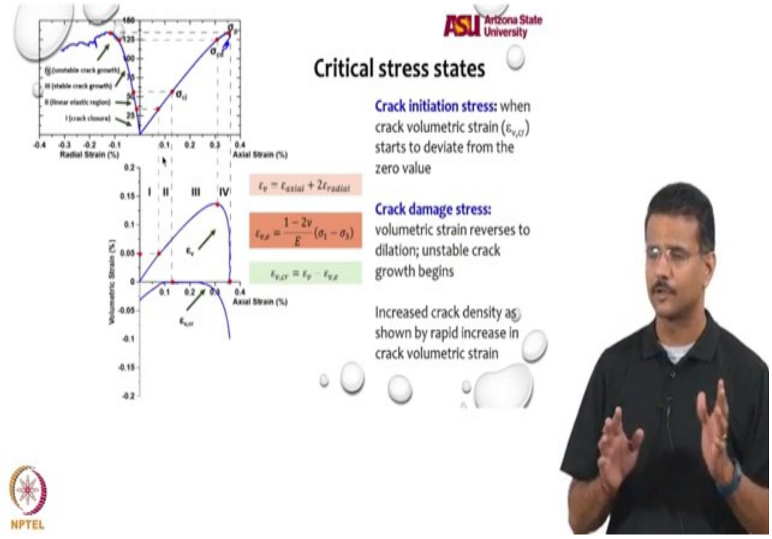
On the right picture you have the response of the mix with 3% fibres. You have much higher radial strain, stress is not dropping anyway close to zero, even at the radial strain of 1% you have about 50% of your axial capacities still left in the specimen. This helps you to understand how you would use this idea of ductility in design and I will show you some tensile pictures also in a while.

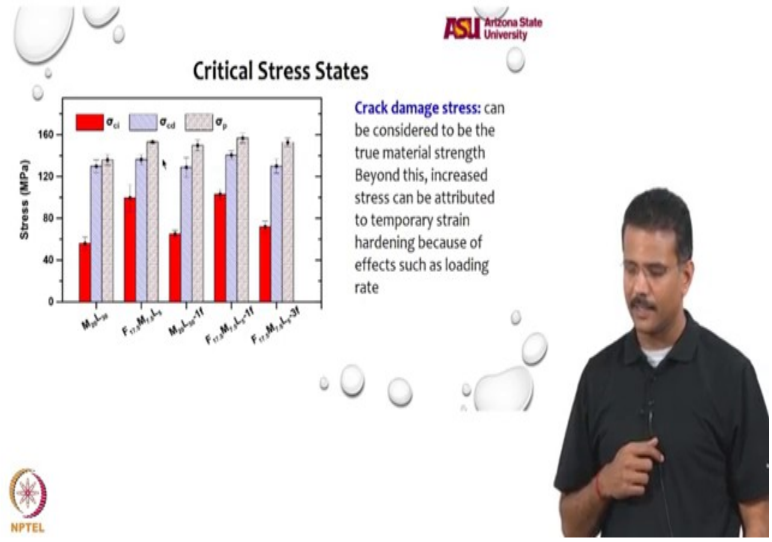
The other thing to notice, if you have 3% fibres typically you expect strain hardening, but we are not seeing strain hardening we are seeing strain softening, that is the trade-off that you make by the addition of aggregates.

The coarse aggregates are known to give you deflection softening behaviour because you create those interfaces around which cracks can propagate and the fibres are not very effective. But if it is a mortar, where the size of the aggregates are very small and fibres can take into account the crack bridging must faster, you will get deflection hardening.

So then the other question is the trade-off, do you want a strain hardening UHPC or are you okay with a deflection or strain softening UHPC. If you want strain hardening UHPC you go for UHPC mortars, if you are okay with some kind of deflection softening, it is not still a brittle failure and you will see there is some post peak deformation, so that is something to think of while you design this mix.

(Refer Slide Time: 6:34)





Now, slightly more different structural aspects to think of when you design this mix. What I have shown in the picture on the left top are the stress strain curves, but marking significant points on stress strain curve based on the strain relationships, so what if the bottom curve are the axial strain versus volumetric strain.

So the response of axial versus volumetric strain, volumetric strain is nothing but the axial strain plus twice the radial strain and the volumetric strain you can use from simple transformation using poissons ratio and the principal stress sigma 1 and sigma 3. The volumetric crack strain is volumetric strain minus elastic strain, elastic is up to 50% of the peak is a random definition.

So if you look at this, it is some very interesting points, you can see the overall volumetric strain on the bottom picture on the positive axis and, you will see the crack volumetric strain on the negative axis of the bottom picture, which is the total volumetric strain minus the volumetric elastic strain.

Now, if I take the point of volumetric strain of 0.05 and come to the volumetric strain curve and then go up hit to the curve on the elastic part then that pretty much is the first part, that is about 25 Megapascal or around that range, where you are actually compressing. You are getting that increased stress because you are closing all the micro cracks within the specimen. You have some positive volumetric strain but you are closing micro cracks in that specimen.

Now, if you look at that point where the crack volumetric strain tends to deviate from 0 and if you take it all the way up, you will get what is called the crack initiation stress which is basically your end of linear elastic region.

Now what happens? Normally we do this all very randomly, you take the slope, you say that slope has kind of look to change and then you take it. In UHPC, you cannot do that because the linear elastic range is much higher because of the very high-strength of material. so you have to actually have this kind of transformation to understand where the crack initiation starts.

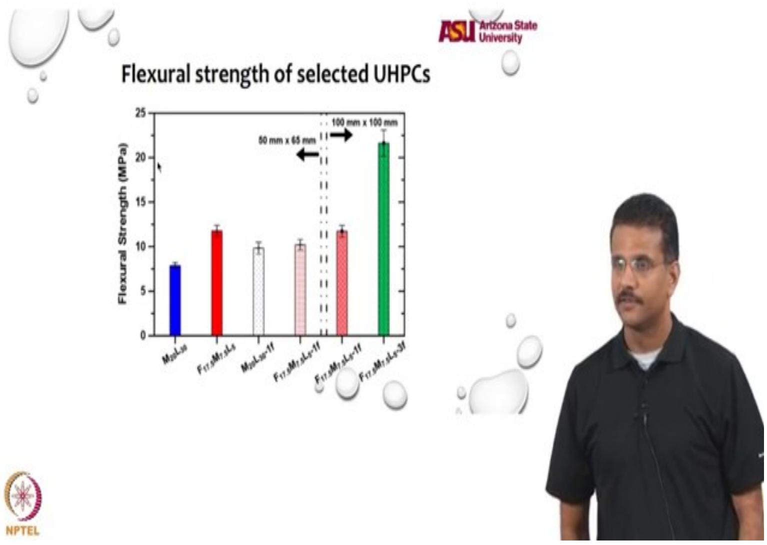
So you can see that at about 50 or 60 Megapascal, you start the crack initiation strain and that is the end of linear elastic regime. So now I am not defining my linear elastic regime as 75% of peak stress like you normally do or linear elastic regime as wherever the tangent intersects or whatever. It is basically based on the idea of strains, the crack volumetric strains, more fundamental, more material response related rather than random fixture of points.

So that is a crack initiation stress and the other important point is a crack damage stress where the volumetric strain reverses to dilation. You see that point marked 4 in the curve where the volumetric strain starts to go back into dilation and there the crack damage stress, volumetric strain reverse dilation and you have an unstable crack growth that is happening.

So that is crack damage stress and we call the crack initiation stress and the crack damage stress as critical stress state for UHPC. Unlike conventional concrete, where you have only the peak strength or peak stress, you actually need these two stress state because you are designing this material to carry extremely high loads and be very durable and crack initiation is not what you want at very early ages. Crack damage or the state where you have starting to have dilation is the end of the load carrying capacity basically.

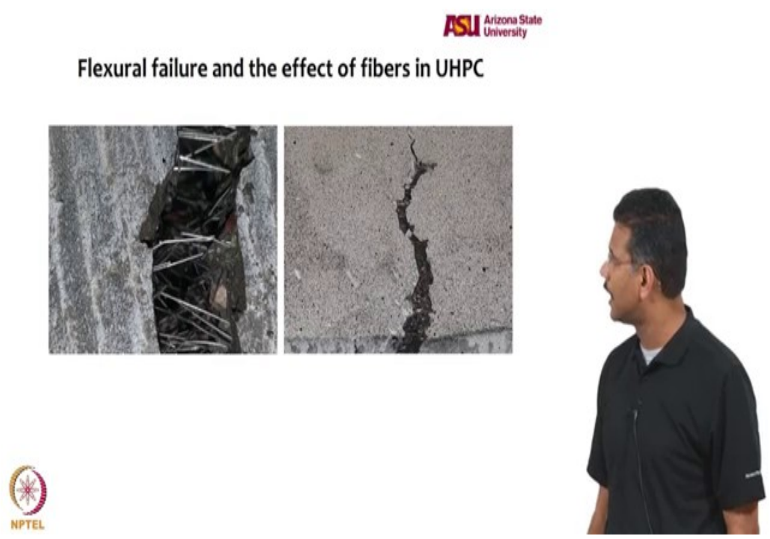
So sigma C, D 125MPa is pretty much what you can theoretically use for design because beyond that what you are seeing are the effects of loading, rapid increase in volumetric strains and stuff which is loading effect, so you do not want to consider that, so crack initiation and crack damage stress and about 25% lower than peak stress, so two different critical stress states.

(Refer Slide Time: 11:14)



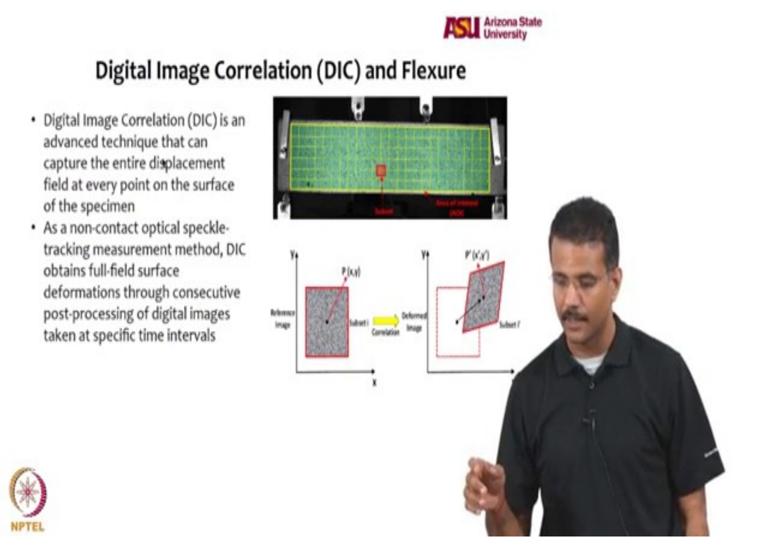
If you look at the flexural strength, 20 to 25 Megapascal flexural strength for these materials, if you have it fiber reinforced extremely high flexural strength.

(Refer Slide Time: 11:25)



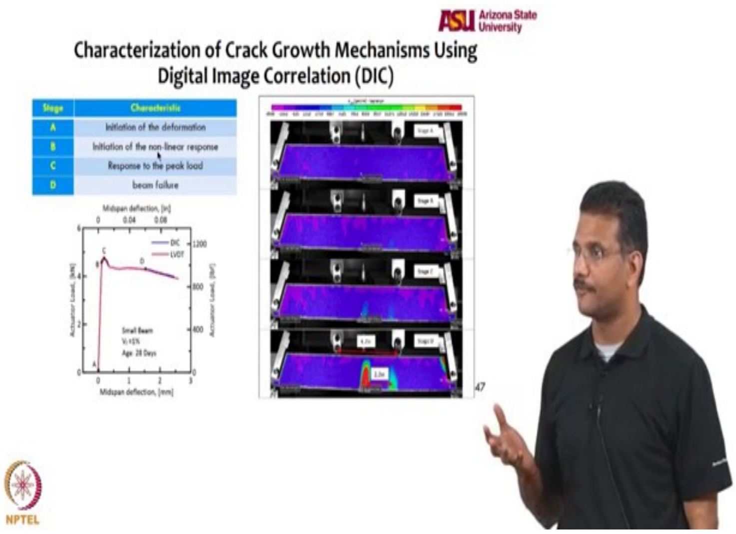
Effect of fibres, again you will see fibres bridging the cracks, a lot of fibres in flexural failure.

(Refer Slide Time: 11:31)



We have done Digital image correlation to actually look at how cracks propagate in these materials. It is a non-contact optical speckling method, you track the surface strains and then based on that you figure out where the crack patterns are and you take images at specific intervals very fast and then you track the movement of every speckle, you do what is called a cross correlation analysis to find out what is actually happening. You can find out the Lagrangian strain fields based on the movement of speckles. Basically it is like surface strains, a point moves from here to there and change in distance divide by initial distance is a strain of that point and you convert that into Lagrangian strain field.

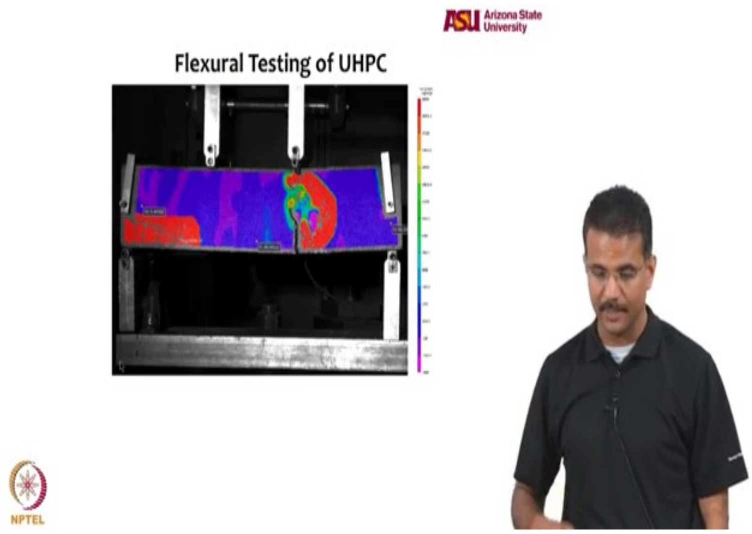
(Refer Slide Time: 12:11)



So just to give you a few important points in the load deformation plot, if you see the top picture, stage A corresponds to just the starting loading pattern, you have almost no strains setup in the material, whatever you see those pink strains at the edges are of the compression of the loading support. It is not really the effect of loads but you just start to see some patterns of strains emerge.

Get to B, B is pretty close to the peak load. You will have some tensile stresses that you see at the bottom and more compression in the top. When you come to C which is at the peak, this is the fiber reinforced specimen that is why you will see a lot of deflection hardening here and this is the fiber reinforce mortar not concrete that is why you see deflection hardening. At C, you will start to see some localisation of strain in the tension zone of the beam. If you now go to point D which is further down in the post peak region, you will see a lot of strains at the centre point of the beam where you likely have formed the crack.

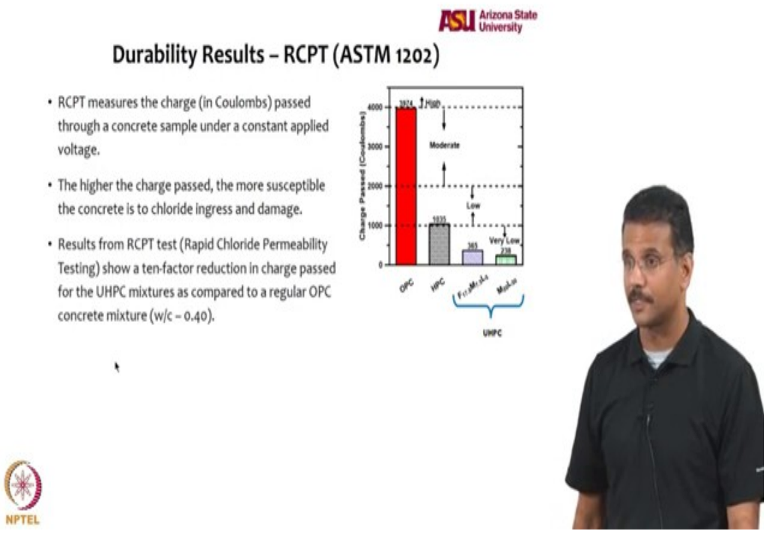
(Refer Slide Time: 13:37)



So I will show you a picture to actually show you how the crack pattern is emerging as a function of loads. So this is all the stages that I have shown you, you can see compression and tension. You see tension building up in the blue, you see those flames coming up at the bottom, so those are all strains. When you starts to see the red, can you see the crack there now, and then that red flame at the tip of the crack is actually a fracture process zone.

So if you, if you theoretically calculate the process zone as some value of strain then you can actually quantify what the process zone is and how the process zone changes with load and you will see random cracking, you will see very extremely high strains at different location. Now, at the edges I would not consider those, those are artefacts but if I just zoom the portion under the load I can now understand how actually a crack propagates in these materials.

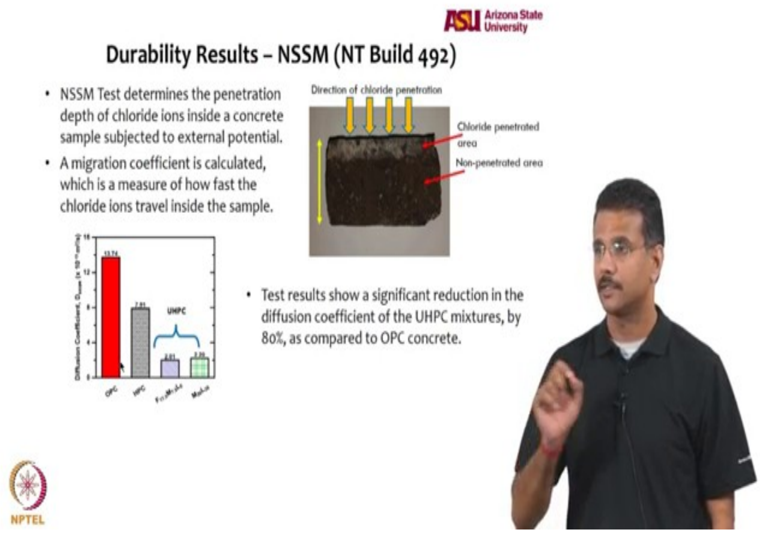
(Refer Slide Time: 14:27)



Finally, a few notes on durability, all of you know what rapid chloride permeability is. If I take typical ordinary Portland cement concrete with a water cement ratio of say, 0.4, you will get a value of about 4000 for RCPT. If you do a high performance concrete which means of water cement ratio of 0.3 (0.25 to 0.3), you will get an RCPT value of 1000. Now with ultrahigh performance concrete, you are getting about 250 and 300, which is a really low value of rapid chloride permeability. I just showed this test and this value, you must have known in your classes that rapid chloride permeability is not something that you should trust all the time. This is because, if you change the pore solution conductivity with materials like silica fume, you have spurious effects with RCPT under high electrical voltage.

So you cannot say that the material is twenty times better if twenty it is times lower , the material probably is five times better. But this test actually exaggerates the effect when you have lowering of pore solution conductivity. But it is better as a quick screening test and not a good test for real performance measurement.

(Refer Slide Time: 15:46)

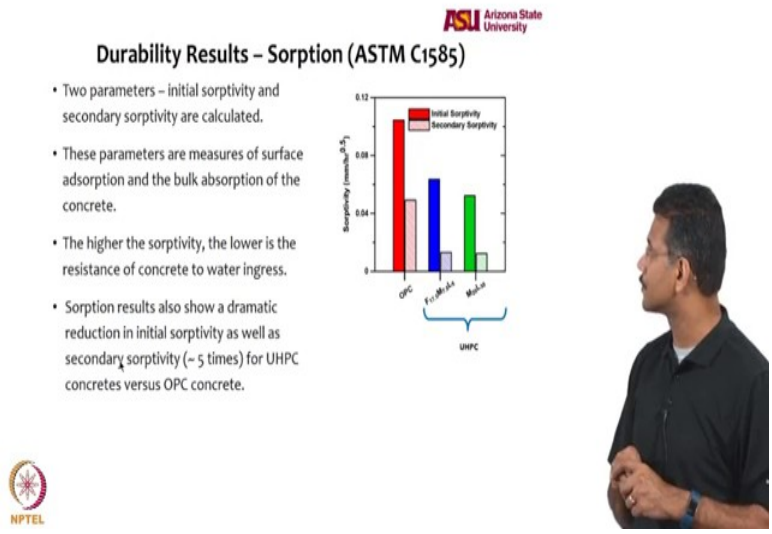


Non-study state migration using NT build 492 is actually quite a bit better test, where your actual application of voltage is lower than rapid chloride test, the concentration of upstream and downstream solutions are also different. So you basically have an electrically induced migration of chlorides, the ions are migrating. But, in any system if you forcefully push ions from one side to the other, there has to be electro neutrality maintained in the system. So for every one chloride that you push into the system, some negative charge should come out of the system. What is a negative charge that can come out of concretes fairly quickly? Hydroxide. So, for one chloride that goes in one hydroxide has to come out and then sodium and potassium and calcium all do the remaining charge balance.

So you carefully control the initial current and initial voltage in such a way that you apply that voltage for 24 hours or 48 hours or whatever depending on the initial current to find out how much of chlorides have gone through, you calculate a chloride diffusion or a migration coefficient. For an OPC concrete, you get about 10^{-12}, for UHPC is about 2 times 10^{-12}, so about 6 times lower migration coefficient. Again, this is accelerated migration, which means you are making electric field push things through, so not a real representation of what is actually happening in the field.

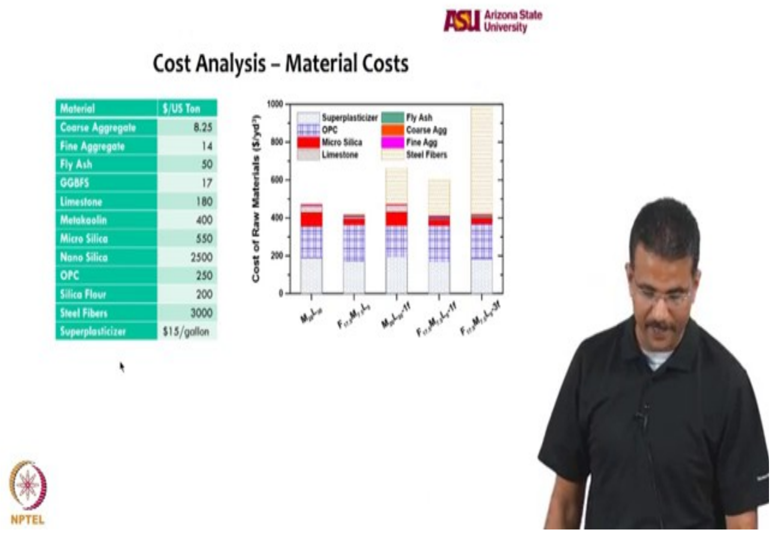
If you do a natural diffusion test by ponding chlorides, you will see that the diffusion coefficient is about an order of magnitude lower. So different test will tell you different levels of where this materials stand, so one test is not essentially an indicator of durability, you have to do a few test to make sure that you get the spectrum of durability enhancements the material rather than just one test.

(Refer Slide Time: 18:04)



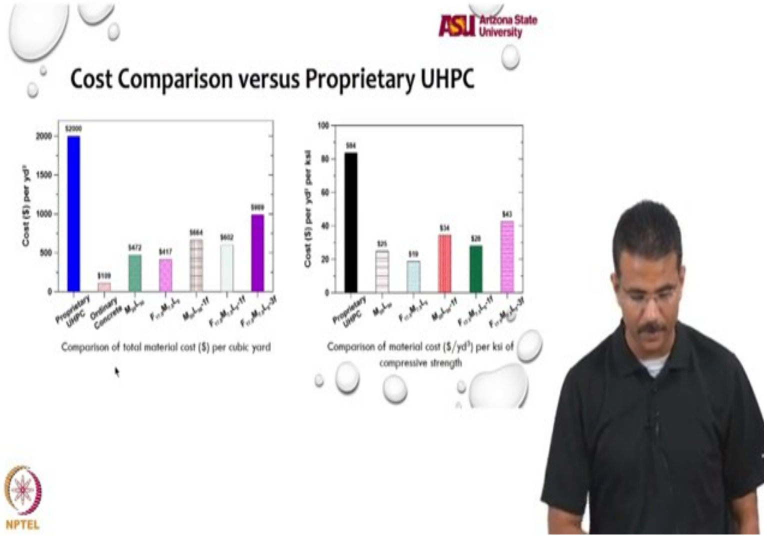
Sorption or water transport also shows similarly relationships. You can do initial sorptivity which is the water intake immediately after you put in for 6 hours and then the long-term sorptivity that you measure for 10 or 14 days. You will see that, the initial sorptivity is governed by the larger pores which can absorb a lot of water early on, the long term sorptivity is governed by some amount of diffusion also through the smaller pores in the material. It is not completely diffusion, you have not probably transformed into completely diffusion control regime but it will give you a fair indication of the size of smaller pores of the the material also which you can see the secondary sorptivity is much lower.

(Refer Slide Time: 18:49)



And then cost analysis, different values in dollars per US Ton, US Ton is 2000 pounds, so dollars per 2000 pounds you see a cost of all those materials. You will see that fibres are the most expensive components with 3000 dollars per 2000 pounds of fibres, a dollar and a half per pound reasonable. Superplasticizer costs about 15 dollars a gallon. So if I use all of that per cubic yard of material for different mixes that I have used, you will see that fibres constitute the highest cost and that is what you pay for the ductility. If you use mixtures without fibres, then the superplasticizer and OPC and the cement are the highest cost, you can see that superplasticizer costs almost the same as that of cement, so it is a very expensive mix.

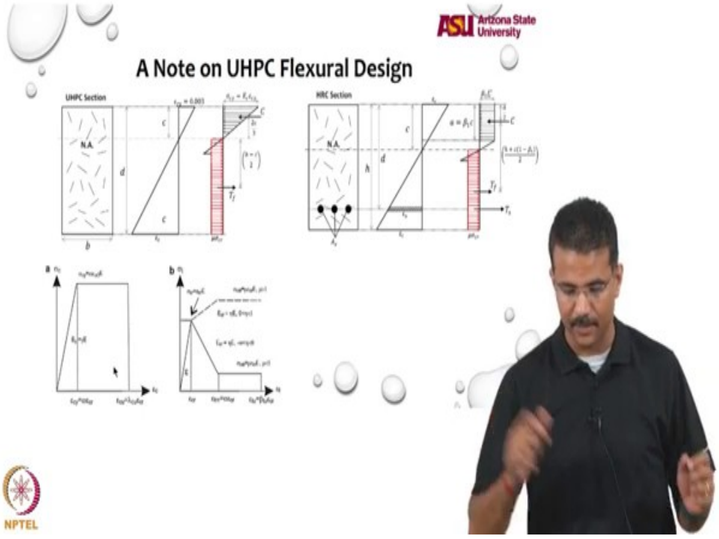
(Refer Slide Time: 19:52)



I told you about 2000 dollars per cubic yard for a conventional commercial ultrahigh performance concrete, so the mixes that we make are anywhere from 450 or 500 to about 1000 dollars. So we are easily able to shave off half the cost and for a department of transportation that is wanting to place 10,000 cubic yards, you can see the amount of cost saving compared to a conventional commercial mixture.

Lafarge makes a commercial UHPC called Ductal and that cost extremely high. Also there are a couple of other UHPC manufactures but a lot of State Department of transportation going towards non-proprietary mixes. So ours is a method that we have proposed as a standard. It hasn’t come yet but we are proposing this as a methodology where you can design ultrahigh performance concrete from first principles.

(Refer Slide Time: 20:57)



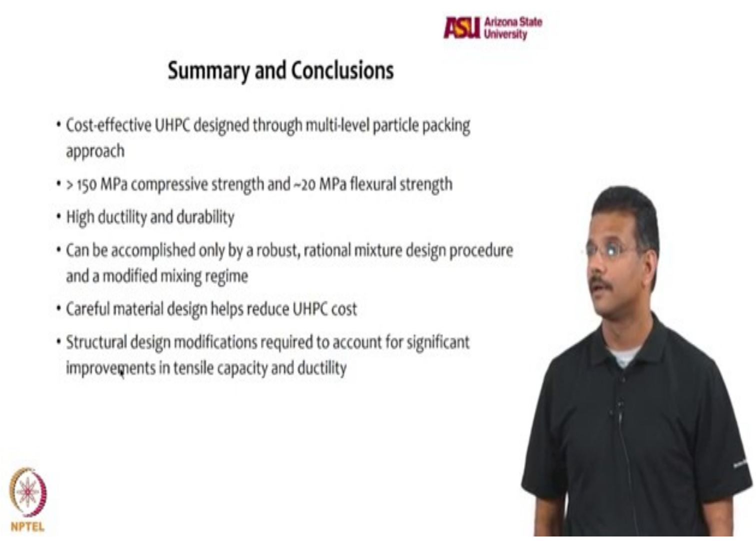
Finally I will give you one little note on design, there is going to be conventionally different design philosophy. The picture that you see on the left is a UHPC section with just steel fibres. The immediate thing to notice is that there is no change in my strain diagram, it is like exactly like conventional concrete strength diagram with 0.003 as the maximum concrete compressive strain. But, I have changes in my stress block. The concrete below the neutral axis is not wasted and a significant amount of tensile stress is carried by it.

So there are two different advantages here. First one is that, if you design a high strength concrete section your neutral axis starts to shift up and a lot concrete below will get wasted depending up on the application of the beam, but now you will not have that concrete wasted. Also, you have a certain tensile capacity of that concrete which can be used beneficially. The second advantage is what we call a hybrid reinforced section where you have a reinforced concrete beam with ultrahigh performance concrete that has fibres.

So you have multiple effects the area below the neutral axis. You have tensile contribution of steel, which is area of steel times the yield stress of steel ($A_S f_y$), plus the tensile contribution of fiber reinforced concrete, which you do a constitutive model for compression and tension. You can see that the constitutive model for compression is a bilinear curve, increase in compression, elastic and then a plastic region. In case of steel, tension it is a trilinear curve where you have an increase in tensile stress, a drop and then a plastic region in the tensile zone and you have different parameters.

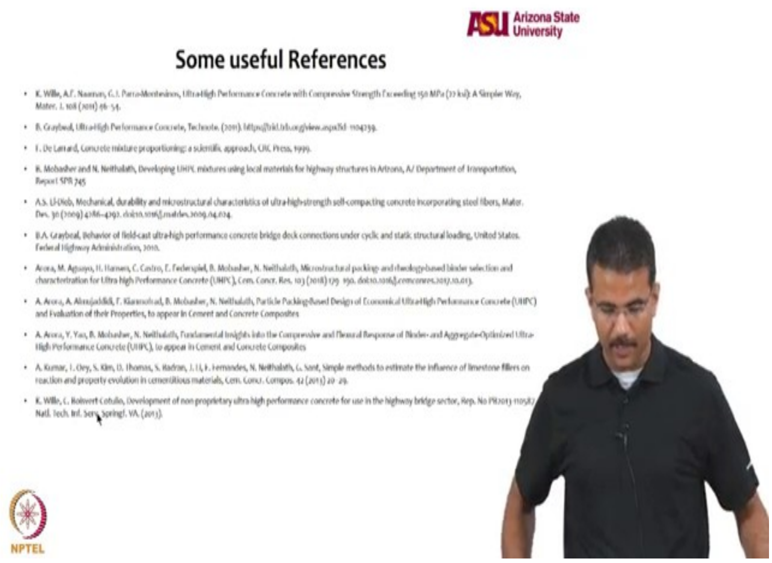
So now with this you can start to design your sections much more efficiently and much more economically, so that you are using quite a bit of the advantage of concrete. Otherwise if you look at the section, if you do not have steel fibres, more than half of concrete is wasted structurally, but here you can start to use some of those advantages. So, the increase in cost can be accommodated a little bit by doing appropriate design strategies and that is a topic for a completely different class.

(Refer Slide Time: 23:24)



So in summary, you have a multilevel particle packing approach both for the paste fraction as well as for the aggregate fraction. Paste fraction is dictated by packing of particles and rheology and the aggregate fraction is dictated by what we say is a compressible packing model but you can use any kind of packing model, you can have very high compressive and flexural strength, ductility is very high, durability is very high, we have done cracking test, cracking propensity is low provided you cure them really well, there are issues with original shrinkage which has to be very well controlled. This property can only be accomplished by a robust rational mixture design procedure, you cannot do a general mixes and procedure using any other standard available methods by which are water cement ratio based mostly. Water cement ratio is basically a secondary parameter here, first is always packing, water cement ratio is that which is required to make this thing flow and fill. You have to have careful material designed to ensure that it is economical and then structural design modifications to account for significant improvements in tensile capacity. It is the tensile capacity and ductility that makes this material very different from conventional concretes.

(Refer Slide Time: 24:50)



Finally I have a bunch of references here for anybody wanting to look at more and there are paper's where we have explained the entire particle packing and mixture design philosophies and some reports from Federal Highway Administration that have explained the advantages of UHPC, where is it used, What is its life-cycle benefits in structural engineering and all of that. Thank you very much.