Transcript Summary:

MR. TAWRESEY: This morning, we went through six different solutions to two trial designs. I'd like to comment that the results were fairly consistent. ASCE's Structural Engineering Institute has recently done trial designs. The results were far more varied. However, the group assembled here are experts. We are not your average structural engineer designing masonry. The results should be closer.

Lets go around the room and get comments:

MR. BROWN: Sure. My reaction is that I'm surprised we should have such close agreement in designs. I think the solutions were more or less all correct. There are three or four issues that we are not in complete agreement on, and I think we have written them down on the carts.

The question that I think we should ask is: Are the restrictions that are imposed on our designs necessary? Is this truly necessary to prevent toe crushing in shear walls, and I would like to hear the opinions of people in this room. They are not going to fix this code. But certainly we need to start thinking about this for the next code cycle. The other issues are matters of just making a decision. Real differences result if compression force is included in the steel when it is not laterally tied. There are a few others.

MR. KLINGNER: The only thing that I would like to add is that when we talk about what do we want to do, it's clear to me that the requirements for minimum and maximum reinforcement are driving us in certain directions in our designs. Did our good intentions drive us in a direction that is undesirable? Are there other directions that we could have gone that would have been better?

MR. DILL: We have gone a long ways to create projects that prevent crushing the toe, and I want to ask the question: Are we creating sections that are now very exposed to lots of tension yielding and subject to sudden tension rupture of the reinforcement?

A second question has to do with walls where their mode of failure is inherently shear rather than flexure. Masonry walls that are 30, 40 feet long by 18 feet high are very common. The mode of failure in the wall would seem to be shear, not flexure. Imposing very light flexure reinforcement is not the right answer for those walls where their failure mode is governed by shear, so I'd like to see us discuss that question as well.

A third question has to do with ductility provisions for in plane and out of plane. I'm very confused as to walls that are loaded out of plane or in plane only. In seismic zones where everything is subjected to its own self-weight, I don't understand how walls are only loaded in one direction. The ductility provisions apply in both directions and the out of plane provision becomes meaningless. It's a requirement that we have included in the code that can never apply. So I'd like to see that issue discussed.

That's what I have.

MR. KLINGNER: The FEMA's commentary to the maximum reinforcement provision does not read to me that the intent is to prevent toe crushing. I am concerned about R values that we are using in that they seem low. They don't seem to have a particular rational basis. I think it needs further investigation.

And last, should we have one criteria for maximum steel or are variations in criteria tied to the selection of R values. We are not addressing how we expect the building to perform, because earthquakes vary. We don't necessarily expect the building performance to be the same for a small frequent earthquake as opposed to a very large earthquake. It appears to me that we need to put something in there to address these situations.

MR. JOHNSON: I do think that we should be aware and conscious that some significant confusion or different interpretations exist between the experts here. If there is confusion here, imagine your, quote, average designer. If it becomes too difficult to do, we don't want to hear someone say, "Well, I am just not going to use it, make it out of concrete." I'm going to reiterate what Rich said. We don't want our good intentions to drive us down the wrong path.

MR. THOMPSON: I would just like to say that so far this has been an incredibly productive morning. I don't want to open Pandora's lid any more than it's been opened. But I have a lot of things I have on my list. But I think they address a lot of things more globally in addition to some specific points.

MR. TAWRESEY: I think you should open the lid a little bit. At least what are you talking about?

MR. THOMPSON: If you look at how we want buildings to perform as a system instead of as elements. Terry had kind of broached the subject. Are we looking only at life safety? Do we want those systems to perform at a given level, and then what provisions give us that level of performance as opposed to giving us a prescriptive requirement. The prescriptive requirement gives us a level that is not explicitly stating. What level is it?

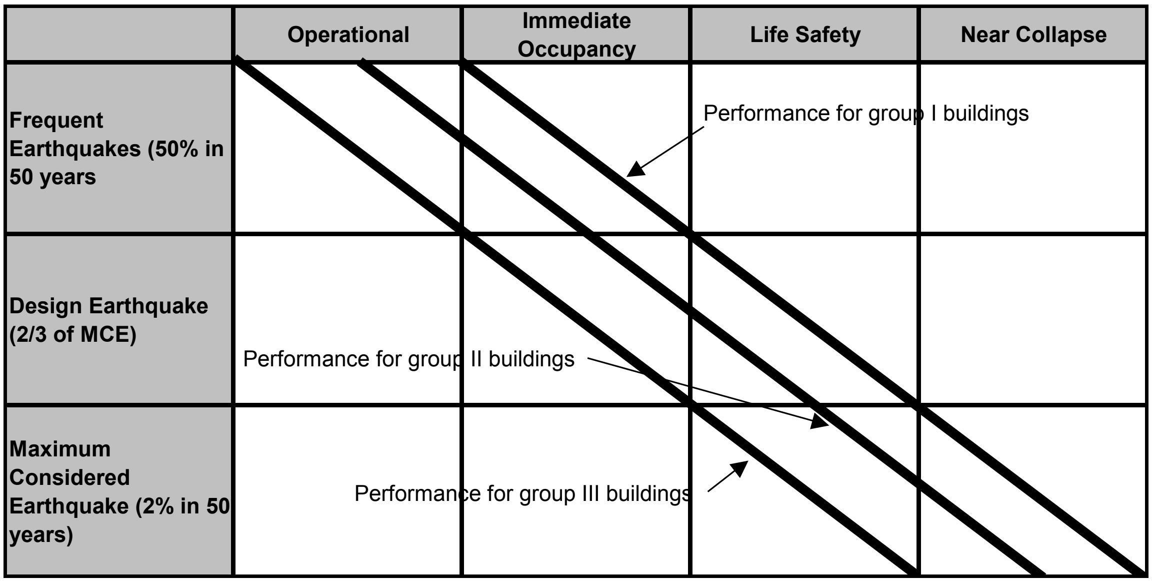
MR. TAWRESEY: So there is a range of performances. We are just looking at one level right now.

MR. THOMPSON: And I don't know if that one level we are all in agreement with.

MR. KLINGNER: It's a level but what is that level?

MR. TAWRESEY: Well, we have the FEMA chart. It is a performance chart, and you should all be pretty familiar with this chart. Where are we on this chart with these provisions?

(*The chart is Figure C1.3 EXPECTED BUILDING PERFORMANCE, Page 8 of the 2000 NEHRP Commentary)*



MR. KANONIK: I'm probably jumping into this later than you all. I have to admit that. But, I'm curious as to what I think Eric and I were talking about. How do other countries' codes deal with this. New Zealand and Australia.

MR. JOHNSON: To my knowledge they do not have a maximum.

MR. KANONIK: They are seismically active, and they have been doing strength design. I don't know if any comparison of their codes to our codes has been done.

MR. TAWRESEY: What about a comparison to the concrete code? The big difference is concrete doesn't include the axial load in maximum reinforcement.

MR. KANONIK: Right.

MR. ELDER: Speaking as a manufacturer, I'm concerned about our ability to compete with other materials and what we have done with those codes has taken away our ability to compete. You take designs that have been constructed, actually built in the past. These buildings can't be constructed any more. And I think that is a real concern. We are concerned from listening to the designer state when he comes to us and asks us what he is supposed to do with the code, we refer him back to the ASD because we don't think that this is really fixed. We think that there are so many little holes in the system that need to be reworked or at least closed that we continually refer them back to a system that is in a direction different than we really wanted to go. I think working stress is always considered to be a more conservative approach rather than less, and strength in my understanding is that you are trying to approach a little less conservative state because you know more what the values are. What are the material properties? What are you really doing with this material so you can narrow down the safety factors? I think the reverse is actually happened to where strength has become too prescriptive and unconservative. I think looking at what Ed Huston says that where ρ_{max} applies you have virtually no ability to add reinforcement with a 1500 psi material.

MR. HUSTON: You can't get to half a percent.

MR. ELDER: That's ridiculous. We just need to be able to use the strengths of the materials. I guess that's the problem plus making it easier for an engineer to design.

MR. HUSTON: Just a couple of things that I will throw out. One is the issues of the confusing nature of the code. How do the designers use it if those of us sitting around the table, many of you have worked on the code, and I've done a fair amount of masonry design, so I feel I should be able to approach it and understand what it means, and we have disagreements in how we look at those things. Some of them are as basic as the gravity load of the Rho max. that Rich pointed out in the calculated, What gravity load, we are supposed to incorporate?

What is d? I use the length of the wall. Rich uses the 80 percent of the length of wall. We can talk about it and figure that out amongst ourselves, but we are not giving the person who doesn't have Rich or I or Terry or Dan or Steve in the room with them any guidance on what maybe should be used there.

Second point vis-a-vis Rho max. I'm going to use John Tawresey's "look out the window". We have got a lot of buildings in seismically active zones with multiple times more reinforcement than this code would allow. And some of those have been subjected to some pretty good earthquakes. How did our masonry do? We did very well thank you very much. But I would like to see what their factors of reinforcement are compared to what we are saying that we should be using. How low a percentage of steel we should be able to use. We have a fundamental disconnect between what our code says and what our environment is. I'm really pleased with the consistency of the answers. I'm surprised. I thought we would be a lot more variability on the board.

The last point is the steel industry is now saying that, after many years of saying that they weren't going to print a new ASD book, they are working on an ASD, LRFD side by side book. And I look at what we see in masonry where we empirical, ASD and strength design and the variety of the answers that we get out of it and the amount of work that you get as a designer that you have to put forth to get those answers. I see another disconnect there. If we want people to use strength design, if we think it's better as Jeff said, if it's really supposed to give us a better answer, and we are should be asking people to go there. We have to look at what disincentives for people to go there. One is that the answer is better. And the other is it takes less work to get there. So in fact none of us did it, and I think maybe Russ should be the one to do it because he has the software. We should compare these trial designs with an ASD solution and see what we have got.

MR. ABRAMS: It's all been said earlier, and I agree with everything, but let me try to reiterate in my own framework. I think this has been very good today for raising many questions, and I think we are a good coherent group. Many of the questions and the issues are comparable to each other. We have got 16 of those issues over there. A few of us have to go to Chicago on Tuesday (BSSC TS 5 meeting) and maybe write proposals for the next cycle and Terry is going to wonder how to do that tonight probably.

MR. KLINGER: Maybe on the way home tomorrow on the plane.

MR. ABRAMS: But one underlying issue is the effect of axle load on ρ_{max}. We can't make the systems work. I thought it was almost already preconceived and/or rigged coming in here. Not that you would do that.

But another critical issue here is why can't masonry be more like concrete? I mean, we don't use axial load for that. I agree masonry is better in this regard, but the impact is people will build a concrete wall because you don't have to put in such a low ρ_{max}. So, should we be philosophical and should we be less rational and be more consistent with that other materials? I don't know the answer to that. But I guess the bottom line is are we building better walls with this method? Do they come out any better? We are paying a huge penalty in reducing the competitiveness. I think we need to have another look.

But one of the fearful things I heard this morning is allowable stress design (ASD) is better. I mean, we are going backwards, guys, if that is the case. If that gets around the practice that ASD is better, we are going backwards. Or just use a lower R factor. Use an R factor consistent with a 1.5 ductility and make it work that way. We should have a method. So there are many questions, many issues, and much of what we have here is based on our wisdom and experience talking around the table, but not a whole lot on test data. Our wisdom is based on test data, but I haven't seen any good test data specifically addressed in these code issues.

MR. ELDER: It needs more research.

MR. ABRAMS: I'm building up to that. I believe if we want to move in a forward direction as an industry that more research is necessary. And we have got the essence of the group here to conceive and design.

MR. CHRYSLER: You know that a lot of where we are at today is reaction to the Northridge earthquake and many of the people at this table came right here after the earthquake. What really boggles my mind is we built buildings 10 to 18 stories without using extraordinary materials. Now we have difficulty building a building half that tall. That really doesn't make any sense to me. That's all I have to say.

MR. BROWN: I think that we need to fix this design procedure because it's forcing us to do things that we know aren't good. I think it's better to spread reinforcement along the wall. It's forcing us to use high compressive masonry. It encourages us to use low yield strength bars. I don't think that is any of the things we want to do. But that is what it takes to make this procedure work. We haven't talked about some of the procedures being apply to elements for which they shouldn't. We didn't get to that in our discussion today. Some of you have heard me say this before, but for those who haven't, a lintel has to meet the criteria in this code. We have got to fix some of the provisions that apply blindly to elements that have no role in the seismic resistance of the system. Before we leave today, I would like to get the collective consensus of this group about the two issues that seem to be the ones that were most clearly misunderstood and that is on compression in the steel.

Does it apply? And also on the piers. Is that really different? Find out what we think it means and what we know it should mean, which might be two different answers. But I think this would be a valuable piece of information to leave here with today. If we can certainly start fixing it that it might even be appropriate to write a white paper to assist engineers in interpreting some of the gray areas.

MR. KLINGER: Russ, you are right about what we are trying to do. We need to be clear on that. I think that we need to make clear that first, these restrictions shouldn't apply to elements that can be uncoupled from the lateral force-resisting system. We have tried a couple of times to find language that would be appropriate. It seems like we can't do it, but that's consistently the intent.

With respect to the crucial shear wall, the loaded in-plane, Steve I agree. I don't know how to fix up a real wall so that it responded out-of-plane only. But for a shear wall in-plane, if we say all right we are trying to get a certain amount of ductility, good. Then we ought to go look directly at the relationship between the R value that we use for design and the ductility demand. Because right now the justification for the critical strain gradient is expressed in terms of drift. Drift is not very sensitive to the R we use for design. And it might be possible to work out a relationship whereas R decreases for the lateral force-resistant system, the gradient becomes less and less critical. That's the new thought I had.

MR. DILL: I vote that we jump on to some of the issues.

MR. WEIGEL: Even with a high R value (with a five or six) and current strain gradient provisions, it would prevent these types of the walls from being designed, would it not? Is that your point?

MR. KLINGNER: I think the R values that we are using now are too low. I think that the C_d value that we end up implies ductility demand and is considerably lower than the R. So with very, very large values of R, it might make it buildable, but we ought to look at that and address it directly.

MR. WEIGEL: I agree with that. I think that is a good thing to do.

MR. TAWRESEY: But isn't it true that if we have an R of 12 we would still have a problem.

MR. WEIGEL: I think Rich has a valid point that we have to look at these R values and relate them to ductility. We could have strain gradient criteria go out the window if we are able to do that.

MR. KLINGNER: If you could have a highly reinforced element with the axial load above the balance point.

MR. TAWRESEY: I'm not sure we need to go above the balance point. We need to get closer than we are now.

MR. HUSTON: Again though even in areas where we have had recent large earthquakes I think we are looking at reinforcement ratios that are 10 times, 15 times, what these equations are giving us. And even if it wasn't the design earthquake -- even if it was only 50 percent of the design earthquake we don't see damage.

MR. KLINGNER: The building is elastic no matter what we do for design.

MR. HUSTON: I have a problem from Concrete Masonry Association of California and Nevada. I think it's a five-story masonry building. It has walls in it with No. 9's at 16 inches on center, and it's an actual building that was built. And I think it's in Northridge or it's in LA, so it went through the earthquake and now we are saying No. 5's at 24 inches on center are the most that we can get in the wall.

MR. TAWRESEY: Well, is there something wrong with a 12-inch thick wall with No. 4's at 24 inches on center, as we have in this trial design?

MR. HUSTON: With 3,000 psi compressive strength block.

MR. TAWRESEY: With 3,000 psi compression strength block. I can't imagine that there is any reason for putting the reinforcement in at all. The wall will never feel it. That wall is so strong relative to that reinforcement, so where we are going? We are going back to unreinforced masonry with seismic design. Just make it thick and strong.

MR. BROWN: With no regard to the cracking mode. In this code now you can have a cracking load that is larger than the ultimate load and that's a problem.

MR. KLINGNER: Well, it's different if you assume that nothing else has happened first like differential movement between a foundation and a wall out of plane or temperature to give you a crack anyway.

MR. DILL: Difference or no difference I think it's wrong to reduce the elastic region of behavior. We have essentially eliminated it. We go from uncrack behavior into yield to potentially rupture.

MR. KLINGNER: Last I think elastic region of behavior is now gone from the building. And I can't believe that is a smart thing to do. I think that it is a really nice zone for these buildings to operate in under all kinds of loading. Taking John's point a bit further, I wouldn't stamp this design either. I just flat couldn't do it. It runs so contrary to my design experience that I would be very uncomfortable putting a design out with 2,000 PSI masonry and No. 3's at 12 inches on center, and I can't believe that is a lasting design.

Part of the disconnect is that we have global behavior looking at the elements of the building. The code looks at individual shear walls. It may be very difficult to bridge that gap. But I think that it is part of our problem.

MR. BROWN: Can different elements of a building be designed with different factors?

MR. KLINGNER: Not according to the current code

MR. KLINGNER: Well, Russ, it can be handled with our existing language. You say that the maximum reinforcement provision applies only to elements that are part of the lateral force resisting system.

MR. BROWN: Exactly the words I proposed in one of my negative votes.

MR. HUSTON: But, Russ, it doesn't necessarily help you. For example, consider lintels, With that language the Rho max may not apply to lintels. But if that lintel is on the exterior of the building, it becomes part of the lateral force resisting system if only for the diaphragm continuity.

MR. BROWN: It doesn't behave in the manner that it is designed. It doesn't have any negative moment capacity or any –

MR. HUSTON: Or compression.

MR. BROWN: -- or any transfer of moment at the ends, so it's just going along for the ride.

MR. HUSTON: Well, it may be transferring high tensions or compression forces from the diaphragm and the moment may be a very small portion of that load case, but a high compression force is like a large axial force.

MR. THOMPSON: I hear what you are saying. I think I agree with you about the lintels, but I think what Russ is really pointing out is that if you have a small little beam over a door, it still has to meet those same criteria.

MR. BROWN: I wasn't talking about the lintels that Mark had in his design. He had continuous lintels.

MR. KLINGNER: But look, Jason, suppose somebody comes with the issue of: do you want control joints on a lintel. It's related to the issue that Ed brought up. If you have a continuous diaphragm cord, if you have done something to isolate that element then the criteria does not need to apply.

MR. THOMPSON: So if a control joint went through a lintel –

MR. KLINGNER: At the end of the lintel jogging through the opening which is our standard recommended practice.

MR. THOMPSON: Different issue. Yes. I see what you are saying.

MR. HUSTON: But we have to somehow in the code tell when they do need to use Rho max and when they don't. It's not as easy as saying it doesn't apply to lintels. Your language, Russ, was better.

MR. BROWN: It's a much more complex issue than that. I think we need to change the conditions for Rho max. I think they are impossible to work with. They are just too restrictive. As long as you are making your neutral axis that close to the compression face and having all the steel yielding in tension with 25 percent more, there is just no way to get any axial capacity.

MR. KLINGNER: One of the points that I think Ed brought up about concrete. You know, one of the things that has already impressed me about concrete design is when we teach students how to do it, we beat into their heads the second week of class that they are never supposed to use a steel area that is anywhere close to the balance area. Then we turn around we make the beam vertical we put axial loads on it. Now it's Winter's rules. They can do practically anything they want up to an axial load limited by say 80 percent of the pure compressive capacity, and all we do is reduce the phi factor. Columns are no less important than beams.

MR. TAWRESEY: There have been a lot of concrete columns that have failed in earthquakes.

MR. KLINGNER: Because they don't have walls

MR. TAWRESEY: And they keep adding more ties, more requirements for ties, and I'm not sure they are getting more ductility because they are not addressing the axial load.

MR. KLINGNER: Right.

MR. THOMPSON: John, if I could diverge for 30 seconds. Russ has brought up about the software. We are about ready to come up with beta review. If -- I would like to offer around the table if you want to review it, I would give you a free version of the software if you want to come up with comments about it.

MR. KLINGNER: Can I have one a week ago?

MR. THOMPSON: I can't make it retroactive.

MR. TAWRESEY: Are there rewards for making mistakes?

MR. THOMPSON: Correct free software. How's that?

MR. TAWRESEY: Free software. Any other general comments? If not, let's start going down the issues that we have identified this morning. I have pasted them on the wall. We don't really have one on that first issue because I think it was taken care of. There was some confusion about section 3.2.5.5, but I think that has been taken care of, so we are done with that.

Does the Rho Max equation include stresses in compression steel. The code doesn't allow this in calculation of the design capacity unless it is tied, so we have a disconnect between the -- I guess the assumptions in the Rho max equation versus the capacity equations. Is that something that anybody cares about?

MR. BROWN: There's not a connection between the published equations and this code. This code is the rule. This code says that there is no compression allowed if you don't laterally tie the steel.

MR. TAWRESEY: But that could be interpreted to only apply when calculating the capacity.

MR. KLINGNER: In fact there is a sentence in that section that says that the stress.

MR. BROWN: Stress in the reinforced --

MR. KLINGNER: Exactly, which implies that you would be using compression in the reinforcement to calculate Rho Max.

MR. BROWN: It may imply that but it doesn’t say it.

MR. KLINGNER: I think the sentence says that if you are not tying the reinforcement, that you don't include it.

MR. BROWN: I don't see any rational basis for including it in Rho max and not including it in capacity design. If it's there, it's there. If it's not, it's not.

MR. TAWRESEY: This is also connected with number 14 on the chart as well. Number 14 deals where compression due to overturning in the Rho max equation. That has to be inconsistent as well, but you can see how big the effect is. Example 2 is not an unusual wall. It happens all the time, where you have the dumbbell wall with shear.

MR. HUSTON: **When you yield curvature, you've got compress in that outside bar. But when I analyze it for the flexural strength, it is in the in tension. I have marbles in my head trying to rationalize what was right and what was wrong. But in completing it the first time, it says yield curvature, yes. You have got to consider that bar in compression if you wish. You can neglect it of course, and that is something entirely different than flexural strength.**

MR. KLINGNER: Because of the difference in the gradient.

MR. HUSTON: You are looking -- but that is what the code tells you.

MR. HUSTON: However, if we do say the compression steel should not be included in the calculation of Rho max, didn't we make Rho max more restrictive.

MR. BROWN: Yes.

MR. HUSTON: So we are sort of going in the wrong direction, you can't win, you can't break even, and you can't get out of the game.

MR. DILL: Can I ask a question of the group? I question the thought process behind having to tie that compression reinforcement in a section where we have gone to great lengths to insure that there is no toe crush. Can somebody explain to me how that bar is going to buckle out of the center of a wall that can't crush? I have so little concern about buckling that one number four in the center of a four-inch wall, I just don't understand. So can someone explain to me the rationale, I guess, as to why we are even talking about buckling those bars?

MR. BROWN: Or as a follow-up, does anyone feel like we shouldn't consider it?

MR. WEIGEL: Well, the question is, if you go past the design drift there is no guarantee that you are not going to crush the toe?

MR. KLINGNER: Well, you might after repeated cycles. Okay. What we are doing is basing the gradient on the maximum drift. Then finding the force in the steel; making sure that force in the steel plus the axial load isn't enough to crush the tow. The question we have talked about before is do we care about toe crushing?

MR. WEIGEL: But if you go past the design drift what happens to the curvature on the wall?

MR. KLINGNER: It increases. It increases the gauge length of the reinforcement. The plastic length will increase also if you have shear in the wall and eventually the bar will rupture.

MR. DILL: Now would you rather have that bar rupture or the toe crush? I would rather have a toe crush than the bar rupture.

MR. BROWN: I would rather have the toe crush.

MR. DILL: We are talking very limited length of the toe crush. When we rupture that steel, first we lose stability of it. Secondly, repairing it is a bloody nightmare. It's very easy to repair a toe crush.

MR. TAWRESEY: Toe crush is a common mode of failure both in concrete and masonry in past earthquakes.

MR. BROWN: Does anyone here think we should not allow compress in that steel?

MR. THOMPSON: That we should not allow compression in the steel?

MR. BROWN: Yes.

MR. ABRAMS: In terms of ASD.

MR. BROWN: And in terms of the design of the wall.

MR. WEIGEL: Without including yield.

MR. BROWN: Yes. Just use the steel in compress and tension throughout. That is what IBC strength has now.

MR. ABRAMS: What do they do for concrete walls?

MR. BROWN: I don't think they do it.

MR. KANONIK: I think that is where the --

MR. KLINGNER: If you have **.05 ASGFG** then boundary limits are restrictive like the columns.

MR. DILL: You are usually containing multiple layers of steel. A whole different ball game.

MR. ABRAMS: And the compression bar --

MR. KLINGNER: And of a boundary element.

MR. ABRAMS: So they are not conceivably running along the wall web.

MR. KLINGNER: No. You will have a mess.

MR. ABRAMS: That's impractical in concrete too.

MR. BROWN: I didn't hear any –

MR. TAWRESEY: It doesn't sound like you have a real consensus.

MR. KLINGNER: Russ, the reason at least for me you didn't. That is absolutely right that I think that the direction that you are trying to go is a good direction. But I'm thinking of this bar that may have yielded in tension due to a previous cycle one way, and now we are coming back. And I know from having watched toes, walls, and experiments that if you have cracking in walls on the tension side and yielding that, now I come back in the other direction, it's very likely that that bar will start to buckle.

MR. BROWN: Especially with a wide crack.

MR. KLINGNER: Because I will have flexural cracks in the toe in the level of the bed joints. And the bar will start to kick sideways.

MR. BROWN: I've seen that in concrete beams with compress and tension steel.

MR. DILL: So this is not including the compress reinforcement or is that only compress --

MR. KLINGNER: Yes.

MR. DILL: Tying it is not going to help. It is a function of the cyclical yielding.

MR. ABRAMS: That's why we should be building unreinforced walls.

MR. TAWRESEY: To respond elastically. So where do we go with this one? Are there further deliberations, further study, what?

MR. BROWN: Most of the people in this room calculated Rho max using compression in the steel.

MR. TAWRESEY: That's one interpretation of the code since the code doesn't say not to.

MR. ABRAMS: It's an equation in the commentary. It is sort of suggested.

MR. BROWN: I'm thinking of the next code cycle if nothing else. We at least want to get it right then. I hope strength design doesn't –

MR. TAWRESEY: Let's try it this way. If somebody comes into your office, a young engineer, and asks you whether or not to include the compression reinforcement in the calculation of Rho max, what do you say?

MR. BROWN: No.

MR. KLINGNER: No.

MR. DILL: Yes.

MR. WEIGEL: No.

MR. THOMPSON: Yes.

MR. KANONIK: No.

MR. ELDER: Yes.

MR. ABRAMS: I don't have a clue.

MR. WEIGEL: Let me tell you why I said no. It is because the way the code is written.

MR. KLINGNER: That's right.

MR. TAWRESEY: I will vote yes but I will vote yes because that is the way I interpreted the code because it doesn't specifically say that in the development of Rho max that you can't.

MR. ABRAMS: And then in the commentary it suggests that you put it in.

MR. HUSTON: Am I reading the wrong section of the code in calculating the maximum permitted as equal ------- I remember graphical axial loads. That's for Rho max.

MR. TAWRESEY: Keep reading it. It tells you to use compression steel.

MR. HUSTON: Tension. The strength of the compression shall be calculated --

MR. KLINGNER: Slow down, Ed.

MR. HUSTON: Stress that is reinforced in the (inaudible) shall be based at linear --

MR. KLINGNER: Right.

MR. HUSTON: So that says that.

MR. KLINGNER: And you have got an equation in the commentary that has for the second term ------ okay a term that implies that you can use the Fs of the compression reinforcement. You have in the code a statement that says you can use Fs only if the steel is laterally tied.

MR. THOMPSON: I put a caveat on my vote, John.

MR. TAWRESEY: You voted yes, didn't you?

MR. THOMPSON: I don't know exactly what I said.

MR. DILL: He voted yes.

MR. THOMPSON: I absolutely agree that I think that is exactly what the code tells you to do. If the question is do I think that is what the intent should be, you get a different answer to that question. Does that make sense?

MR. TAWRESEY: Yes, it does. But okay. Let's try it this way. Well, I think we already got this answered, but I could have posed the question what does the code say and I think we would have gotten the eight to three.

MR. ABRAMS: The code is MSJC?

MR. TAWRESEY: Yes.

MR. ABRAMS: Not also picking up the ------ because it's FEMA's commentary.

MR. WEIGEL: It's not in our commentary in MSJC.

MR. BROWN: You don't have a leg to stand on.

MR. ABRAMS: For those of -- by going to a resource document they would be confused.

MR. BROWN: Well, there is a difference. There is a source document for -- IBC source, so that doesn't have a limitation on the compression. So that would be a different reason because that preceded this.

MR. ABRAMS: Yes. I think in any case that should be in the commentary.

MR. THOMPSON: Could we take another vote where we can ask how we interpret the code and then what we think the code should say.

MR. TAWRESEY: Let's try, should the code require the compression steel be included in the calculation of Rho max?

MR. BROWN: Yes.

MR. KLINGNER: Yes.

MR. DILL: Yes.

MR. JOHNSON: Yes.

MR. THOMPSON: Yes.

MR. KANONIK: Yes.

MR. ELDER: Yes.

MR. HUSTON: Yes.

MR. ABRAMS: Yes.

MR. TAWRESEY: That's easy then. This problem is solved.

MR. KLINGNER: Actually you meant permit?

MR. TAWRESEY: No I meant required. I switched it to required. So it's a prescriptive equation. Everybody ought to get the same answer.

MR. WEIGEL: Then you are suspending any engineer judgment in using the code.

MR. TAWRESEY: I don't think with a prescriptive requirement such as Rho max you need to worry about engineering judgement.

MR. WEIGEL: I can certainly read it right now.

MR. HUSTON: Number 2 is a different issue. It's not Rho max.

MR. TAWRESEY: You are right. But we did get an answer.

MR. BROWN: If you want to know what you said, she can read it back. Never mind.

(Break taken.)

MR. TAWRESEY: Okay. Item number 1. I think it was concluded that the force in the compression reinforcement should be included in the Rho max calculation. And that's the answer. Number 2. is that we have identified that the IBC allows compress steel to be counted without ties and MSJC does not. So do I understand that IBC is going to use MSJC.

MR. HUSTON: It goes away.

MR. TAWRESEY: So do we care?

MR. KLINGNER: If one cares about that one would have to introduce a code change to affect the way that the MSJC is referenced by IBC. One would have to say do this except you know with this and this and this exception.

MR. TAWRESEY: Yes. But the way it's tracking is once MSJC.

MR. BROWN: USD is now in the IBC but there are a lot of caveats that go along with it. There might be an interpretation.

MR. TAWRESEY: There might be an exception.

MR. KLINGNER: Somebody might --

MR. TAWRESEY: Okay. IBC compression steel allows stress without ties.

MR. BROWN: What does it matter in determining how much axial load you can put on.

MR. TAWRESEY: We already decided that.

MR. BROWN: It's kind of nonsense to allow it in one place and not the other. It's just inconsistent but one could argue at -- it's an extreme condition so you shouldn't allow the compression.

MR. THOMPSON: You are looking at just walls.

MR. BROWN: When you are just designing the walls, it's PM diagram the fact that the steel -- it doesn't matter much.

MR. THOMPSON: I'm not agreeing with you, but I do distinguish between columns and those types of things.

MR. BROWN: Well, in a column it would be more significant because you are talking about bigger steel areas.

MR. DILL: As a practical matter doing a calculation. It's almost harder to leave it out.

MR. KLINGNER: Yes.

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MR. DILL: You end up trying to figure out what is a positive stress and what is a negative stress. Let's just count it

MR. BROWN: Okay by me.

MR. TAWRESEY: So it means that MSJC will have to be modified.

MR. BROWN: And modified on its next cycle.

MR. TAWRESEY: Okay. Everybody agree?

MR. KLINGNER: I need to abstain from that.

MR. TAWRESEY: You have a political problem?

MR. KLINGNER: Yes.

MR. TAWRESEY: Do you want to confess to it?

MR. KLINGNER: I'm the incoming Chair. I just -- it wouldn't be mine.

MR. TAWRESEY: Okay. That could be a problem.

MR. KLINGNER: But I'm going to encourage the MSJC to move in the direction of what we agreed on in number two to consider it seriously.

MR. TAWRESEY: Okay.

MR. THOMPSON: I guess it's a good point of clarification, John. Are we looking at trying to formalize the modifications to the 2,000 IBC here?

MR. TAWRESEY: No, I wouldn't.

MR. THOMPSON: If we are at the two thousand and five MSJC for two thousand five thumbs up.

MR. TAWRESEY: Okay. Number 3. Is the shear failure of P-2 critical?

MR. BROWN: What if that wall fails in shear?

MR. TAWRESEY: Right.

MR. KLINGNER: We found out that it was not critical for design. We had some perhaps a greater design shear on it than we really needed. But I wouldn't feel comfortable with P-2 failed in shear. Now I have some kind of a soft story. You know with the remaining walls as P-1 and P-3 --

MR. DILL: I'd love to jump in on that because a soft story cannot be an issue related to one wall. A story is a building issue. And if this wall exceeded its design shear strength, I would expect that this wall would still exert some shear resistance for the building, but it would start unloading. And in this particular building it happens just across the corridor there is another wall. There is another wall there that is going to take a bunch of this shear. And there are walls six or seven other walls on that floor that it is going to distribute to. I think we get -- we scare ourselves to death when we think that anytime a single element undergoes failure we have a weak-story-type of situation and we conclude that the building is going through a weak-story failure. I think we sold ourselves way short. We sold the building way short. It carries us into things that I think are –

MR. HUSTON: Plus keep in mind before you even get here in the building code, you have had to determine if you have an irregularity in the building and do something to address it. That's in Chapter 16, you know. If you are going to develop a soft story, if you got so little wall at that time failure of one wall will develop a soft story, you figured it out before you get to this point in the design.

MR. KLINGNER: Did we have a problem with this wall in design?

MR. BROWN: No.

MR. TAWRESEY: The driving design is not shear. It's flexure.

MR. KLINGNER: Right. That's why it's an issue.

MR. BROWN: Isn't that the one that had a greater code capacity than it needed and as a result it had to have that M of 1.25, jack up the shear by a factor of 2.5, and then John asked the question, well so what if it even failed in the shear? But I think that is more than covered by the code by saying you have to design it for 2.5 times the shear we calculated in the first place. So really we should be discussing something else.

MR. KLINGNER: Okay.

MR. TAWRESEY: Let's go with no action.

MR. KLINGNER: No action.

MR. TAWRESEY: Okay. We had one that we took out. We got that one resolved.

MR. KLINGNER: We ought to indicate how we agree on that. **You should factor by the alpha M_n to the M_u.**

MR. TAWRESEY: Is that commentary material?

MR. BROWN: It would be good commentary material, yes.

MR. TAWRESEY: So that should be added to the commentary. Okay. Number 5. Rho max does not apply to P-1 and P-3 because they are piers. Do we want to leave the escape clause in for piers?

MR. ABRAMS: Are you asking how the code now reads? I could think of a perforated wall.

MR. BROWN: It says "beams, piers, and columns."

MR. ABRAMS: Really?

MR. ELDER: But not shear walls.

MR. BROWN: 3.2.4.

MR. ABRAMS: I only have where it talks about max and steel. It says for walls, for columns and for beams. That's what you are reading?

MR. BROWN: All right. And I am reading 3.2.4. Then it says "design of beams, piers and columns" --

MR. KLINGNER: It does not reference about the -- my opinion is, John, it ought to apply to piers.

MR. ABRAMS: -- equations and 3.2.4, they apply to walls.

MR. KLINGNER: Then you go on to the wall system later.

MR. BROWN: Then it refers back to this shear stuff when it talks about the walls.

MR. KLINGNER: You have walls in 3.2.5 and 3.2.6.

MR. TAWRESEY: So first question, does the code Rho max, does it apply to piers?

MR. KLINGNER: As the code is written, no.

MR. TAWRESEY: Okay. Code no. Should it?

ALL: Yes.

MR. BROWN: I'm not sure that I agree that the code says no now. It's just whatever you want to believe it says.

MR. TAWRESEY: Except Russ. Exception noted.

MR. DILL: Let me ask the group a question.

MR. BROWN: I'm not saying yes. I think it's not clear at all.

MR. DILL: I'm not sure I understand something then. P1 and P3's overwhelming contribution to the lateral system has to do with their tension and compression capacity. So my question to the group is, what contribution does flexural ductility of those piers have to the ductility of that system? I don't think there is any chance of a flexural overload on that pier. I think there is a much more likely chance of a compress overload, a tension overload, due to coupling effects going on there, but I think there is practically zero chance you are going to a crack mode into that pier that is going to force it into the --

MR. BROWN: It's a required drift.

MR. DILL: Its drift is limited by everything else in that line. It's limited by all of the shear walls that are there. Primarily all it's getting is tension or compression from seismic.

MR. BROWN: But it's still going through the drift.

MR. DILL: It's going through the drift.

MR. BROWN: As I understand it, that is what you are designing against even though it's primary tension, and it's got to be able to withstand that drift and still do its job as a compression and tension member.

MR. ABRAMS: In the spirit of the Rho max, you should also limit the amount of the reinforcement in the vertical piers such that the two priers conform to strain gradient over P1, P2 and P3 with the n of 5.

MR. DILL: Right. But that's a different calculation than considering each pier.

MR. ABRAMS: But you would still be limiting the vertical steel in the same fashion, in much the same fashion.

MR. KLINGNER: It would look like the calculation for problem one.

MR. ABRAMS: Yes. Back to what you said earlier, the same wall.

MR. HUSTON: However, once again going back to the other parts of the code that these provisions have to interact with, think about the situation where you took wall P-2 out and you had to discontinue a shear wall and you have enough walls elsewhere.

MR. ABRAMS: No. P-2.

MR. HUSTON: No. P-2.

MR. ABRAMS: Then you would be screwed.

MR. HUSTON: It's built all the time.

MR. ABRAMS: Well, it wouldn't be much of a wall.

MR. HUSTON: Think of the situation where the RCJ hotel where you have got the column and beam, you have got a shear wall on top of it. You have got an area for a conference room or a function. Okay. The other parts of the code -- chapter 16 is now going to tell you that those end piers, the P-1, P-3 piers, have to have more reinforcement in them because they have to be designed for 3 Rw/8 to avoid having an axial failure due to the P-delta effect. So if we are saying the reinforcement has to be less, but the strength has to be much more. Then we have a disconnect.

MR. KLINGNER: One way of handling this Number 5 is to pretend that we knew what we were doing. That we were much smarter than we ever would give ourselves credit for. That when we were drafting those provisions, we looked ahead and we saw the difficulty that would be associated with trying to apply the Rho max to piers. And so we decided in our admissions that it would be better to apply them to the walls, not the individual piers.

MR. WEIGEL: Would your argument not apply to columns if you want it to apply that to piers? So would you be in favor of removing the restrictions (Rho max) for columns also?

MR. DILL: I'm just pointing out that there are elements in buildings that are subjected to different types of load from earthquakes. And some of those things are -- they are elements that are governed by shear behavior. There are elements that are govern by tension capacities as well as flexural capacities, and when we overload a building, we may overload any specific element in any of those types of loading situations.

So we have to think about the fact that we can't just wave a flexural wand at all of this stuff and make it all go away because unfortunately that's not how our buildings work. So here we are talking about a pier, that if we overload, I contend will fail by outright tension. Appling a flexural ductility provision to that pier, you know, applying a ductility provision doesn't really make sense. It has different problems than that.

You know, by the same token taking a wall that is 50 feet long and 10 feet high and applying a flexural ductility provision to that wall doesn't make any sense either. Its failure mode is shear. So we should be asking ourselves will the wall that this shear critical, what is our desired reinforcement package for that situation to keep that thing behaving as well as it can.

I can contend that our flexural provisions will work contrary to the best condition for that wall because we are going to be saying, take all the vertical steel out. I contend that is going to fail in shear. That's not what I want to do. I want to hook it at the end and do all of the stuff that we do, and that's the wrong prescription for that condition.

Here's another one. So, yes, a column would be included in the Rho max., if it's coupled into the lateral system. It could be another example of an element that is really not going to be governed by its flexural behavior.

MR. HUSTON: Then you are going to more than half a percent for a column.

MR. KLINGNER: But to get back to Terry's question, and your responses, take the second problem, beginning here P-2 is not there, - so essentially you have a big wall trying to get overturning into two columns. Messing with the longitudinal reinforcement in those columns except on the tension side is not going to give you desirable behavior. Can you take out or put in all of flexural reinforcements that you want and you haven't done anything. Flexural ductility is not where it's at. You would have the same issue where your looking at columns where you would have an infill frame trying to get base shear out -- into a batch of spindly columns and blowing the columns out in a compression failure at the base.

MR. THOMPSON: If you -- they don't apply to piers, it didn't appear to apply to pier. It wouldn't apply for columns.

MR. WEIGEL: Why are we leaving piers out of this or including columns or whatever?

MR. ABRAMS: I would think it would apply whether they would be a wall or a frame because what you do for the --

MR. WEIGEL: How would you apply it for that wall where you have two piers?

MR. ABRAMS: Perforated wall and the system as much -- as we were saying about 10 minutes ago, we had a lot of wisdom when we wrote it this way because we did. But you want to make the system work such that it has the drift capacity, right? You don't want the individual pier necessarily to work by itself, but you want this system to work.

MR. TAWRESEY: It applies to individual elements in the code, and that's the way it's going to be applied to all elements.

MR. ABRAMS: Well, to get it to consistency wall because it talks about the walls which are resisting elements as well as column and beams.

MR. TAWRESEY: So take out the columns.

MR. ABRAMS: You could do it selectively to make that system work and treat your spandrel beam that Russ is worried about accordingly, so I think we are talking about apples and oranges with walls and beams in the same sentence.

MR. WEIGEL: So you want to take beams out?

MR. ABRAMS: I would say lateral force resistance elements.

MR. BROWN: Some beams are.

MR. ABRAMS: Then the rest with a ductility of five times. A strain is associated with the component. The wall happens to be a component of a lateral force resistance system.

MR. KLINGNER: You could take a drift criteria or a ductility criterion, apply it to the system, the lateral force resisting system, and abstract from that what the corresponding demands would be on the component and we would come to the conclusion that P-1 and P-2 **-- I'm sorry, -P-1 and P-3 have --**

MR. ABRAMS: **So make the axial --**

MR. KLINGNER: **But you --**

MR. ABRAMS: Do you write that in the code or do you expect the user to pick up on that as a way to pick up on the criteria? We are micro managing it here. We are trying to tell them what type of ductility to put on a component to get a system level desired action.

MR. ABRAMS: If you had a frame, would you want to make a beam capable of this ductility? I guess you could.

MR. WEIGEL: Capable of what?

MR. ABRAMS: Capable of going 5xn? Well, coupling beams but maybe not the walls of that story.

MR. THOMPSON: So if we had some kind of drift check in which we are looking at whatever our system had, and look at how that corresponded to each element, then require that each element conform based on what we expect it to do as a function of ε_y.

MR. KLINGNER: The five ε_y is how to go.

MR. ABRAMS: Exactly. I mean it transferred over to a frame system.

MR. KLINGNER: Right.

MR. ABRAMS: Particularly for weak beam strong column you need to have it in all of the columns and all of the beams.

MR. KLINGNER: Right.

MR. ABRAMS: And yet we've got it everywhere.

MR. KLINGNER: Yes.

MR. HUSTON: We are probably not going to solve this today.

MR. JOHNSON: This is a deep problem that is not going to be solved.

MR. TAWRESEY: Well, you are bringing issues on the table that are telling you that maybe it's too early to have a Rho max. Maybe we are not ready for a Rho max applied to elements.

MR. KLINGNER: I think it would be very difficult to write something consistent with Rho max elements but there is, and let me check this, I think that I'm hearing around the table general agreement with the concept of taking required behavior for force resisting systems, then trying to translate that in some consistent manner into requirements on elements and corresponding reinforcement limitations.

MR. WEIGEL: I agree with you, Rich, that is what we are trying to do. But we are trying to take performance criteria and turn it into prescriptive requirement. It's going to be a lot of work. It's going to be a tough thing to do.

MR. DILL: Just as an example of what is involved, Ed's comment on the code requirements on those elements, say we look at a center shear wall, and we have those two piers at the bottom, what the code's intent in telling you to do is to take the hinge out of those two areas with over strength, and push the hinge up in the system elsewhere. That's what they are trying to get you to do.

But how correct is it? Is a smart approach to that problem? If so I think you tie it down really well. Make sure that this shear wall above this, it continually works well. All of that implies that somebody is taking time to identify the inelastic regions in these designs and designing them correctly. And that's in combination to where we are right now, where we try to design and apply ductility. It's everywhere and I think we are getting trapped.

You started out by saying that we want those piers to be ductile. I'm not sure we do. I'm not sure we want those piers to be ductile. We may want them to be strong.

MR. KLINGNER: That's a good -- good intentions driving us toward a design for that entire wall system.

MR. DILL: Yes

MR. KLINGNER: The less vertical steel we have of P-1 and P-3, the worse off we are going to be. We are going to have more rocking at the base. We are going to have rocking piers.

MR. DILL: On these little spindly legs.

MR. HUSTON: This is sort of off the subject but not really. The graphs I showed earlier lead me to a real concern with the Rho max equations, and I know that this is heretical. You can burn me at the stake later. But I have a code change in the IBC that says (you don't have to listen.) -- it says, that -- essentially replace the Rho max with the 97 UBC Rho max with an axial component. I don't know if that will go anywhere, but my hope is that it will let people use the strength design provisions and not get trapped into some of these traps with MSJC. While MSJC considers what to do about the next cycle. So just -- I just want you to know that I'm there, and if I need to be tarred and feathered on the way out of town, I'll live with that.

MR. TAWRESEY: The proposal is only for the strength design.

MR. HUSTON: There are two of them. Actually one envisions that the code change to replace 2108 entirely with chapter three. The other one envisions that it's not successful and one will be withdrawn depending on what happens with 2108. So --

MR. TAWRESEY: Is there a proposal for Rho max for ASD?

MR. HUSTON: There is a proposal for Rho max for ASD.

MR. TAWRESEY: Fifty percent of balanced with an axial component as well.

MR. HUSTON: Yes.

MR. TAWRESEY: What are the chances for passage?

MR. TAWRESEY: I don't know. I haven't heard what MSJC --

MR. THOMPSON: I suspect that it's going to pass. The chance that it's going to pass as is is slightly less, but there will probably be some modifications.

MR. KLINGNER: What is the maximum reinforcement limitation that you are proposing?

MR. HUSTON: The minimum is prescriptive. I would be glad to show you the change.

MR. KLINGNER: Is it what we worked on at MSJC that didn't get in on time?

MR. HUSTON: I didn't get that.

MR. THOMPSON: It's Rho balanced.

MR. HUSTON: It's Rho max times f'm instead of f y instead of f_s because of concerns of the different material over strength.

MR. KLINGNER: Actually that will work. What we showed in developing this is that if you propose a limitation that is based on a Rho balanced for strength design. It actually will be quite consistent for allowable stress design.

MR. THOMPSON: The only limitation that is very different is the limitation on ASD for high seismic activity.

MR. TAWRESEY: Haven't you again lost the big picture that we talked about earlier. The issue is the system ductility and not the element ductility. If this goes through, you put the element ductility requirement onto ASD.

MR. THOMPSON: To be honest with you, John, this change is political drench.

MR. TAWRESEY: What does that mean?

MR. THOMPSON: If we didn't make the change, there would have been a change made for us by someone else.

MR. HUSTON: ASCE 7.

MR. THOMPSON: ASCE 7.

MR. TAWRESEY: ASCE 7 wouldn't have anything to do with this.

MR. HUSTON: They said that if there is not an ASD maximum reinforcement limit, then the R values get slashed.

MR. TAWRESEY: I see.

MR. HUSTON: Big time.

MR. KLINGNER: If we did that to them we would be improperly interfering with what their purview is.

MR. HUSTON: You are absolutely right. We are being whipped.

MR. THOMPSON: With a capital W.

MR. KLINGNER: If we mess with them.

MR. TAWRESEY: Who is “them”?

MR. KLINGNER: ASCE 7. The way they are messing with us, we would be roundly criticized for it. We would be whopped.

MR. DILL: Why don't we put in our own R values? Let's just take whatever they give us, and we will rescale them coming into the masonry chapter.

MR. THOMPSON: That's what is going on right now in the IBC. They do not correspond.

MR. WEIGEL: Where do those values come from?

MR. THOMPSON: They were all scaled up.

MR. HUSTON: I'm afraid I am responsible for some of that because when we saw -- this was the Masonry ad hoc of Oregon -- when we saw the R values, when we saw how much our base shears were going to go up. I wrote a proposal saying it should be different. It turns out that there were enough of those proposals that CRSD at the time had a special summit occurred and they invited all of those people who had made proposals to headquarters, and we were successful in getting at least the special reinforcement R value changed. We were successful in changing it in those 12 cities across the country. We are going up, concrete is going down, wood is going down.

What is the deal? And they bought it for special walls. Jim Harris argued vehemently against it and made us change the reinforcement bar spacing for immediate walls. They said they would take it down farther.

MR. WEIGEL: Is that when it went from 10 to four feet. I wanted to know when it went from 10 to four feet.

MR. HUSTON: It happened in response -- we are going to have an R value of one to two for those intermediate walls, if those spacings didn't change, so there again we are being dictated to.

MR. TAWRESEY: But from what I have seen today, the Rho max restriction may be much more significant than a drop in R.

MR. HUSTON: Absolutely. Absolutely. Shall we go to Number 6? We are going to be here a long time.

MR. TAWRESEY: Are we done with five? It's a good conversation. Item Number 6 is shear allowables for walls with seismic tension due to overturning. The example was wall P-1 and P-3. we have tension and we have shear. I don’t believe the code address the combination of tension and shear.

MR. BROWN: It does have a term in there that you could use as a negative P. If you use a negative value for P, it's addressing T. It's reducing it for the tension.

MR. TAWRESEY: Use negative P. Is that for the commentary?

MR. ABRAMS: It doesn't say you can't use a negative.

MR. BROWN: No, it doesn't.

MR. ABRAMS: It doesn't use --

MR. HUSTON: That's what needs to be decided.

MR. WEIGEL: I'm not sure it's that simple, just reverse the sign. You've got enough tension for cracking and engaging the vertical reinforcing.

MR. BROWN: Well, you have got a wall that's has tension, and in this case the wall has more tension than compression, but you've still have a length of wall that is in compression.

MR. WIEGEL: Why wouldn't you go to the other wall to add shear resistance? Would you do that?

MR. BROWN: You are going to do it with this same equation using the positive value for P. It will come out in the wash in this particular problem.

MR. DILL: I don't think there is any way to get a tension into a pier without coupling. And, therefore, whenever you are coupling it, you are getting compression elsewhere in the system.

MR. WIEGEL: Yes. That's fine. Why don't we just design -- why don't we count on zero shear for the wall in tension and put the 2.5 P on the wall in tension.

MR. KLINGNER: But since the factor applied to the P is the same, it doesn't really make any difference. All you are doing is losing stuff on one side gaining on the other. Okay. Now, you can also make -- you can make the point if you want to drive yourself nuts in the calculation. You can try to look at the relationship between the shear that it's taking. You will take more shear in the compression pier. It becomes even perhaps something that is -- right. As John will show us it's a real mess, so I think that it's simpler and just as valid to not worry about the tension in the pier due to overturning because you have another element that's going to have its capacity increased.

MR. ABRAMS: If you want to complicate it more, think about the distribution between the two shears, and one in axial tension doesn't attract a lot of shear because it's got open cracks, and the one compression is attracting more.

MR. WIEGEL: Put it all in the one pier.

MR. ABRAMS: You have all seen diagonal cracks in the end piers but not like this because of that effect. Even though they are stronger in shear they crack, but that is too complex for the code. Just think if it's an elastic distribution consider each one okay. You have covered it.

MR. HUSTON: As a design are you saying when I do my design yes it's okay to use a negative P when I use a negative I have V_m and V_s also?

MR. ABRAMS: It is a function of negative P.

MR. WIEGEL: What happens if the negative P overcomes the other part of the equation?

MR. THOMPSON: You get less than 0.

MR. ABRAMS: Then you have negative shear strength.

MR. TAWRESEY: All right. What do we do with this? Is it further study?

MR. BROWN: It's okay as is.

MR. WIEGEL: I suggest we write some commentary language.

MR. TAWRESEY: Okay. The next issue is number 7 and is the requirement that 80 percent of the stiffness be wall stiffness. Does this requirement eliminate the common building system that combines frames and shear walls.

MR. HUSTON: Code section 3.1.3.1 on the first page.

MR. TAWRESEY: There is a class of buildings that is fairly successful that has a shear wall core and a frame around it.

MR. WIEGEL: The World Trade Center?

MR. DILL: That is not a good example.

MR. TAWRESEY: It appears that the provisions say you can't have this type of building.

MR. WEIGEL: My interpretation of the provision applies to buildings where there are masonry columns in addition to masonry shear walls.

MR. KLINGNER: So when we read 3.1.3.1, it says that at each story level, at least 80 percent of the lateral stiffness shall be provided by lateral force resisting shear walls, and along each column line at a particular story level at least 80 percent of the lateral stiffness shall be provided by shear walls. So the design that John drew up there doesn't meet the second criteria.

MR. TAWRESEY: Right.

MR. HUSTON: But could meet the first criteria.

MR. KLINGNER: But could meet the first criteria. And depending upon your definition, I could also meet the second criteria, and if I made some of my wall elements a little bit longer in the plan. I mean, all I would have to do is make them long enough so they would not be a column.

MR. TAWRESEY: This seems to be a Chapter 16 type issue, not something that is in the masonry code.

MR. ABRAMS: Steel frame with a block wall. Why would it be in this chapter?

MR. TAWRESEY: Right. It seems like it's overly restrictive to me.

MR. DILL: It's also very difficult to interpret because what drew me to this problem is the fact that we had these procedures that are an integral part of the overturning wall. We took them at 80 percent of the stiffness of that wall. It went away but geometrically they are piers, so I had real key parts of the lateral system where they were classified as piers. What do I do? From a shear standpoint more than 80 percent of the shear was in the center wall but they had an enormous amount of overturn, so what does that mean?

MR. KLINGNER: What I propose that we do on this, John, is to ask the seismic people at MSJC to look at this.

MR. TAWRESEY: MSJC to review and delete.

(Laughter.)

MR. TAWRESEY: Okay. MSJC to review. Next is number 8. The question is "what is d?

MR. KLINGNER: L_w for in-plane and one half of the wall thickness for a wall out-of-plane.

MR. TAWRESEY: What does it say in the notation? Doesn't it say the distance from the extreme compression fiber to the centriod of the tension reinforcement?

MR. HUSTON: Excuse me. It isn't necessarily one half the thickness of the wall for out-of-plane if you had two curtains of steel.

MR. KLINGNER: With a single curtain it is.

MR. HUSTON: It's to the bar, even for a single curtain.

MR. KLINGNER: It's the conventional to place a single curtain at the center of the wall.

MR. BROWN: We don't have a definition for d. The equation is not in the code or the commentary, so we need to address it now to clarify what we mean by d.

MR. ABRAMS: It should be defined from the extreme compression fiber to the extreme bar.

MR. KLINGNER: Right.

MR. ABRAMS: Yes.

MR. TAWRESEY: But to the centroid of the reinforcement has been traditional definition and is the definition in ASD. So do we know what d is?

MR. KLINGNER: We know what d is.

MR. ABRAMS: Yes. We don't know what d is.

MR. KLINGNER: For the Rho max -- for the Rho max equation it's not in the definitions.

MR. TAWRESEY: Yet. In plane. It's L_w.

MR. BROWN: That's what put this on the board to begin with is that issue.

MR. BROWN: If you were using a discreet analysis.

MR. ABRAMS: But in the definition of Rho max, it's over bd. We are talking about the fourth digit here.

MR. TAWRESEY: Well, we are not because if you take the definition, if you used distributed steel, and you take the definition Rho over bd where d is to the centroid the value is a lot different than Rho over bL_w.

MR. KLINGNER: Or can you take from the compression fiber to the extreme layers of steel?

MR. TAWRESEY: Yes. You could.

MR. KLINGNER: Which is fine.

MR. BROWN: It is four inches less.

MR. KLINGNER: It doesn't make any difference.

MR. HUSTON: No.

MR. WEIGLE: Well, it depends on how thick it is. It would have to be a column or a pier.

MR. TAWRESEY: So how do we define Rho? Is equal to a A_s divided by bL_w .

MR. ABRAMS: You take a sub --

MR. HUSTON: No. The actual term in Rho max.

MR. KLINGNER: It doesn't matter whether you use -- you can either use d equal to the distance to the extreme steel or can you use L_w. In fact, the terms stay the same.

MR. TAWRESEY: Only because of the small difference in this example. But, you can't use the d definition in the code.

MR. KLINGNER: Right. So if we put it in the equation we will have to put in either a change to the definition of d or use bL_w.

MR. TAWRESEY: Where does it occur in the equation?

MR. BROWN: It's not in the code or the commentary.

MR. HUSTON: What brought this up is several of us did it with the numbers.

MR. TAWRESEY: Can we clear it up?

MR. ABRAMS: When you calculate shear it has the definition, d_v which is defined as L_w.

MR. TAWRESEY: Are there some places where this has to be reviewed in the MSJC? It should be on your list.

MR. BROWN: d_v is the actual length in the direction of the shear.

MR. TAWRESEY: I've always had a problem. Which direction is shear? I don't understand that.

MR. ABRAMS: That would be the full thickness of the wall in the direction that you are measuring it.

MR. TAWRESEY: Both horizontal and vertical. So what is the direction?

MR. ABRAMS: Oh, you mechanic guys are all alike.

MR. TAWRESEY: Exactly. Design a building for an R of 1.5. Does it work? I think probably it does.

MR. BROWN: Well, in the case we tried to make it work for piers P-1 and P-3. If you use that R of 1.5 instead of 3.5, your earthquake load will go up by three and a half to one and a half. Is that right? And your allowable axial load which is related to P max doesn't go up enough to compensate for -- doubles the axial load.

MR. TAWRESEY: But you are not limited on axial load or the amount of the reinforcement anymore. We only had three number four bars in pier for tension.

MR. BROWN: There is a section three through tower structure design less one point five, and then you use a ductility ratio of two instead of five. There is a big goof in this section, Jason.

MR. THOMPSON: There is a huge goof, I know.

MR. BROWN: Because it says if you have an out-of-plane wall and you use an R of 6 or so, you design it for a ductility ratio of 1.3. But if you use an R of 1.5, you design for a ductility ratio of 2. So it went backward.

MR. WEIGEL: Yes. The intention there was to give the designer an alternative if they didn't want to do an elastic analysis but it didn't quite work out that way.

MR. BROWN: But it was good for the shear wall. Was it intended to only apply for shear walls?

MR. KLINGNER: It was intended to only apply for in-plane.

MR. BROWN: Okay.

MR. TAWRESEY: Well, I think I do remember this n of 2. The number changed the day after the discussion in San Francisco. The first day's discussion on this was about buildings being design to respond to elasticity, which meant that there - if it responded elastically the R is 1.5. Right? That's not complicated. The n of 2 showed up the next day. You agree the the n=2 should go away?

MR. KLINGNER: No. Because when we talked about that the justification for using what the strain of two times yield is, it was related to the expected uncertainty of the design earthquake. But if you are really designing the structure to be elastic and you don't have ductility, then you ought to take a higher load even a higher penalty because that's one of the things that protects us against uncertainty in the earthquake. It is the fact that the capacity is determined by steel, which we can predict. So the two was put in there intentionally.

MR. TAWRESEY: But that's the problem in the loads, not in the material. And if you design an -- unreinforced the masonry building at 1.5 ---.

MR. KLINGNER: Can't you design a reinforced one. All loads have uncertainty?

MR. TAWRESEY: That's why you have the 1.4E in the load combinations.

MR. BROWN: Be careful what questions you ask or they will start screwing with unreinforced.

MR. KLINGNER: To get on because I think that -- it seems to me that we are looking at a combination of issue Number 9, issue 10 and issue 5 that this is a components versus lateral forcing systems. But it seems to me that where we were headed with this, we hadn't worked -- we hadn't worked enough design examples, one of the things that I do intend to ask all of the subcommittees to do is don't propose a change about a draft design example. We want to really see where this stuff puts us. But it would be very reasonable with simple kinds of systems to have the coefficient on the ε_y in the term for maximum steel to tie that to C_d, the ductility demand not the R. If we say that an elastic response corresponds to a R value of 1.5 because of over-strength and energy dissipation and soil and foundation system da da da, then if I am using -- if I use an R factor of 4.5, for instance, that corresponds to a ductility of 3 and what I ought to use in the Rho max equation is what I have now and that is 3.0 ε_y , something like that. Difficulty is how to make it so the engineer doesn't have to spend 40 years figuring out what he or she is going to do before they actually do the design.

MR. DILL: Aren't we asking them to identify their inelastic mechanism?

MR. WEIGEL: Critical inelastic.

MR. DILL: To make all the elements strong enough to deliver the loads to those inelastic elements.

MR. KLINGNER: And make those elements ductile enough to do what we say the building is doing with the forces doing it.

MR. TAWRESEY: And the performance criteria is not losing the vertical load carrying capacity rather than no damage --

MR. WEIGEL: Well it depends on --

MR. TAWRESEY: I understand.

MR. WEIGEL: But you are right.

MR. TAWRESEY: The performance criteria has to be brought into it.

MR. WEIGEL: Could we take a straw vote just so I can get a feel at the table?

MR. THOMPSON: If you had a reinforced system that you are designing for response of 1.5, and you design the elements of resisting the forces or just going along for the ride, to be able to take the ductility, would you impose a Rho max on that system? Is that question clear?

MR. JOHNSON: Yes. It is to me.

MR. KLINGNER: Run that by me one more time.

MR. THOMPSON: For a system, a masonry system, to have an R value of 1.5 and I designed the non lateral load resisting elements to withstanding the ductility whatever you are imposing on those elements, would you impose or is the maximum reinforcement ratio necessary?

MR. ABRAMS: According to the code, yes.

MR. WEIGEL: Maybe I missed something somewhere, but we only have one choice of 1.5. What we are talking about is designing an elastic system as reinforced for some reason -- we decided to even though it corresponded to an un-reinforced system R.

MR. THOMPSON: This is not what the code is telling us but more of a general global kind of question. Would we reinforce the system that was designed to be elastic.

MR. BROWN: It seems to me that we are taking a double penalty if we do I hear what Rich says. But my --

MR. KLINGNER: My answer to Jason's question would be if I have a sufficient knowledge of the uncertainty of the load, there is no need to impose an additional restriction on Rho max.

MR. THOMPSON: You felt confident with earthquake loads. We will treat it like wind.

MR. KLINGNER: And I could handle it either way and I think that maybe getting to the heart of what I sense your question is, if we decide that the structure is going to behave elastically, and I think Oh, I'm not so certain about the loads, is it rational to try to protect myself by controlling steel or by controlling an over-strength ratio? It makes much more sense to me to mandate an over-strength ratio as we do for other kinds of brittle failure like shear. If I want to put in No. 6’s in every cell, I could do it. Would you agree?

MR. THOMPSON: I guess that's kind of the question I am asking.

MR. KLINGNER: Okay.

MR. HUSTON: I would be comfortable with that.

MR. DILL: With that within reason. I mean I don't believe in eight-inch walls with number nines in every cell. So I think you have to be able to confine steel to some extent.

MR. KLINGNER: No splitting.

(Mr. Elder leaves the room.)

MR. WEIGEL: According to FEMA, we only have one wall that has an R of 1.5.

MR. TAWRESEY: What's the highest type of wall?

MR. WEIGEL: That's a special wall which has an R of 3.5.

MR. TAWRESEY: So if you design for a special wall, you follow all the requirements of special wall and use R of 1.5, then you can escape Rho max. Let me say it again. If you designed with all the special requirements, okay, so you are using reinforced masonry and use an R of 1.5, then you escape Rho max. Can we have agreement on that?

MR. KLINGNER: You might require some kind of an over-strength like requiring that toe crushing be over-strength like we have for shear, you know, that if you -- if you don't have a capacity design, then you have to design for 2.5 over-strength or something like this. Okay. But certainly controlling Rho max doesn't seem like the correct way to address uncertainty about the loads.

MR. TAWRESEY: Okay. Possibly some over-strength factor. So now we have agreement on this. Anyone disagree? Okay. Everybody says yes. That should be fairly easy.

MR. TAWRESEY: All right. So we lump 9 and 10 together so we probably ought to have some discussion on the criteria, the drift criteria versus ductility. I think we got some discussion on this already. Do we need to amplify this further?

The way I look at maximum drift delta, there is a little bit of elasticity and we have to go there with ductility. How much?

MR. KLINGNER: Somewhere about five, okay.

MR. TAWRESEY: So we have a one to five ratio. So now what are we talking about? I'm talking about other options, other ratio’s besides one to five. Trade some ductility for strength.

MR. KLINGNER: Yeah. What we tried to do, the five to one or the factor on ε_y comes from two sources, both of which gave the same answer. The one in the FEMA commentary that says put on the drift of 1 percent, assume a hinge length of that's about equal to the length of the wall. Then that gives you the strain gradient in the wall. The same strain gradient as five times yield on the reinforcement at the end of the wall. Okay. The 1.3 was back calculated so that you would have about the same answer for an eight-inch wall -- concrete masonry wall fully grouted. Dan and I did it. And where was it? San Diego. That was the place where we met, and they were setting up for a rock concert?

MR. KLINGNER: And they were doing sound checks.

MR. KLINGNER: And then we also found out that the 1.3 was legitimate because in Ron Mayes' tests of out-of-plane walls. The hinge length for the out-of-plane walls is huge compared to the thickness of the wall. Then at the end to try to protect people who were doing a design that would be essentially elastic, we came up with the third provision. So we could make that explicit. We could say you take the system, you figure out the C_d that you expect the system to experience, okay, and you can get the C_d that you expect for the system by taking the R value that you have used and multiplied by one and a half.

MR. TAWRESEY: So this is the C_d. This overhead elastic C_d and use that to define that. Is that what we want to talk about?

MR. THOMPSON: The huge disconnect that I have, we are looking at huge $5\varepsilon_y$. I don't have a whole lot of confidence.

MR. BROWN: I'm just thinking about the designer that has got so many things to comply with. And then he is doing change C_d and Rho max, and it's really going to be complicated.

MR. TAWRESEY: P-1, P-2 and P-3. We would put the entire shear in P-2 and we could do it in about five minutes.

MR. KLINGNER: Yeah. Right.

MR. TAWRESEY: I'm not sure that still isn't as good as --

MR. KLINGNER: Throw all the shear into the web.

MR. TAWRESEY: That's the way those elements have always been designed. That was standard office procedure.

MR. KLINGNER: But, John, so we can be perhaps be even more clear here, there is a -- that number that you would take call that R divided by one point five.

MR. THOMPSON: Well, it's inversive proportional.

MR. TAWRESEY: Right. It's the force to -- the total elastic force to the actual capacity elastic forces the R and C_d, so I'm not sure you could do this. Because you don't know. This is a system force deflection curve.

MR. THOMPSON: Are you distinguishing between --

MR. KLINGNER: Well, it's your expected system of ductility.

MR. DILL: Instead of using C_d, how about if we use the elastic drift times C_d, so what we are really saying we have this inelastic piece?

MR. TAWRESEY: Same thing. It works out somehow we are just conceptualizing.

MR. DILL: If you have that system, then that should come down.

MR. KLINGNER: Yeah.

MR. TAWRESEY: There are parts of the code that limit elastic drift.

MR. KLINGNER: Right. But I guess what the other piece that we haven't talked about here that convinces me that it's better to tie this to ductility is that for the overall problem of getting R for a system, that is a very flexible system. You might decide that R should be based on controlling drift because of the second order effects on collapsed and also ductility deformation for a wall type system that we are dealing with. R is never going to be controlled by elastic drift. Elastic drift is always pretty small. What will control R is ductility, so for us to have the criterion on drift, we are trying to solve the problem the wrong way. The drifts in our buildings, wall type buildings are not sensitive to what we use for R. It would be if we had a steel frame. It could be with no other drift limits. So for other reasons, I think this is the way to go.

MR. TAWRESEY: You lost me.

MR. KLINGNER: Good.

MR. THOMPSON: Just to summarize, I think everyone agrees that some scaled version of C_d.

MR. BROWN: Some variation in the ductility requirement relative to elastic strength. Which was the intent of this provision that was pointed out a minute ago, but make it like a sliding thing not a discreet either/or but anywhere you want to be.

MR. TAWRESEY: The way I understand it, is as you increase the elastic capacity, the R is smaller. Which means that there is less ductility required. So you trade strength for ductility.

MR. KLINGNER: Right. Which is logical.

MR. TAWRESEY: Which is logical. It doesn't matter how you get there. You are still trading the elastic response. You have more elastic response. You need less ductility and vice versa.

MR. DILL: I also think that as you have more stiffness, you have less inelastics demand, so I think we are not just talk about C_d. We talked about C_d as modified by our calculations, so if I was standing there looking at an elastic drift associated with scaling that up, it's a very tiny number. My inelastic demand on my elements is very low. And I think the building that we just designed is an example. We had a .3 percent drift. That's considerably lower than the rotations that we looked at when we developed these ductility provisions.

MR. KLINGNER: Okay. So not looking at just the ratio, but given two buildings with the same ductility demand one with a much smaller yield displacement than another one, you favor that second because in deformation is less, I would agree.

MR. HUSTON: One last comment. Chapter 16 defines design categories and the seismic design categories tell you what type of wall you have to use. So for seismic D, E, and F, you have to use a special shear wall with an R of 4.5 so how are we --

MR. KLINGNER: You have to use the special shear wall. You can use a smaller R if you want.

MR. ABRAMS: There is no lower R limit.

MR. TAWRESEY: If you use a lower R you should get some relief on Rho max.

MR. HUSTON: And if you use a lower R --

MR. TAWRESEY: Okay. Number 11: Calculation of Rho max was correct. I think we have a bunch of different methods here. But I think we pretty much covered those in previous discussion. Which one do you want next?

MR. ABRAMS: What are the choices?

MR. TAWRESEY: We have IBC-MSJC FEMA and --

MR. DILL: The Dill plateau. (The compression reinforcement yields).

MR. THOMPSON: If we have a system R, shouldn't we model those?

MR. TAWRESEY: The realization here is only the n changes in the calculation of Rho max. But the formation of equation is the issue.

MR. KLINGNER: Well, if the code that you are using says that you should let the steel that is compression yield, if the strain is greater than the yield, then I think that is what it should be.

MR. TAWRESEY: Yes. I think we said that previously.

MR. ABRAMS: If you answered a question about systems versus members, this is a good point. This is getting back to the Rho max based on the members.

MR. TAWRESEY: Within that context this is a simple question.

MR. ABRAMS: All four of those methods are the incorrect way.

MR. TAWRESEY: Therefore just get rid of Rho max.

MR. ABRAMS: Moot issue.

MR. TAWRESEY: Okay. Number 12. Distributed versus concentrated steel location. I think what we learned this morning is the methods currently in the code favor concentrated steel. You get better capacity and higher Rho max, so if these methods begin to be used regularly, the result will be concentrated steel.

MR. KLINGNER: This is taking us in an undesirable direction.

MR. TAWRESEY: So this is an issue of where we are headed.

MR. BROWN: It's kind of a boundary element. It's just a structural element on the ends of the wall.

MR. HUSTON: That's the way you typically design an ASD with end steel but are ignoring all the steel in the wall.

MR. TAWRESEY: What do we call this? Trim steel? Is everybody happy with that term?

MR. ABRAMS: I think that is descriptive.

MR. KLINGNER: I think concentrated is better.

MR. TAWRESEY: We are talking about shear walls. In the industry we have always called it trim steel.

MR. KLINGNER: If you just leave it concentrated, everybody will know.

MR. TAWRESEY: Okay. So there is another awareness issue. The next awareness issue is a new market for ultra low strength steel. Number 13.

MR. BROWN: And negative steel.

MR. TAWRESEY: And negative steel.

MR. ABRAMS: And you need a new specification on how the steel can be low enough.

MR. TAWRESEY: Does anybody want to give Hank Martin a call?

MR. ABRAMS: And how do you err on the side of conservativeness? You make it weaker.

MR. TAWRESEY: Yes.

MR. ABRAMS: Maximum. You can't exceed it, but you can be below it or right on the money.

MR. TAWRESEY: Well, no. There is still some grade 40 steel around. I think can you get it.

MR. ABRAMS: Is that a 40 or a 55?

MR. TAWRESEY: Oh, probably 55 but you probably got the 1.25 factor anyway.

MR. HUSTON: Going back to that number 13, I don't know, maybe this is a pipe dream, but my hope is that as all this gets discussed at MSJC, then some of these issues, a lot of these issues, get cured at the same time, and what we maybe see on the next MSJC is a radical departure from what we are seeing in these examples.

MR. ABRAMS: Kind of like a good wine that takes years.

MR. HUSTON: I'm not sure we have years. I mean, it goes back to --

MR. TAWRESEY: It's hard to get industry support when they are not doing any masonry work anymore.

MR. HUSTON: It's bigger. It goes back to what Rich said earlier. If we were doing to them what they were doing to us, as a designer in the next issues of codes we are going to be looking at the codes where we are mandating to use strength design. We can't use the one-third stress increase. ASCE 7 is the only load combination in the code, which is the direction that people are trying to go. You are penalized heavily for doing ASD.

MR. HUSTON: On the other hand, at the same time if we penalize ourselves for using strength design, then, you know, we don't have to worry about it if it's in the designer's guide. I don't think we have years. That was my way of answering your question. I don't think we have years to solve these things. I think these are very important to practice.

MR. TAWRESEY: Number 14: Rho max, should it include overturning or not?

MR. DILL: This is on the element.

MR. HUSTON: We talked about that.

MR. TAWRESEY: All right. Done. We have got a couple more and then we are done. Tension rupture of reinforcement with low Rho max. There is no limitation to the cracking that maybe at a higher stain level than the rupturing strain of the reinforcement. There is a provision for out-of-plane.

MR. DILL: I think even more than that, I think if we overload these members we have a choice in what happens. And if we reduce the reinforcement to a certain level, what will happen is rupture. I think we are better off extending the ductility of the element and then asking for a toe crush rather than taking steel rupture as an ultimate result. This is because steel rupture is a drastic performance issue and a horrible repair issue. So I think –

MR. ABRAMS: But how do you promote yielding in the bar before toe crushing and then on the other hand try to prohibit the fracture?

MR. DILL: You bring it down. $5\varepsilon_y$ is too close to rupture. You are just out there to where you have to let something else happen first, and the only other place to release the energy and unload the system is compression toe crushing.

MR. ABRAMS: You have a fused concept where you don't want fuses to break. You want the fuse to yield but not break.

MR. DILL: How do we want it -- we are in a -- we want a long inelastic region. The question is when we come to the end of that inelastic region, what do you want to have happen?

MR. KLINGNER: We are not close to the end of the plateau yet. $5\varepsilon_y$ is about halfway even with today's steels. It's about halfway on the plateau.

MR. BROWN: It's about strain hardener. It's about when it starts.

MR. DILL: So you guys all agree that we want to avoid toe crush at all costs?

MR. TAWRESEY: Well, according to -- according to my numbers with a 12-inch wall and number 3 bars, when that wall cracks, you are not going to get any ductility of steel. When you crack that wall, it's not even going to see the steel. The steel is just going to break. We have seen it out-of-plane tests and the provisions are therefore in the code for out-of-plane.

MR. KLINGNER: I hear two distinct issues here. The first one is would this be okay if you started off with a bond breaker at the base of the wall before the earthquake came. Okay. The second one is, what happens if M_u or M_n is less than M cracking?

MR. TAWRESEY: It seems like we ought to look at a provision similar to the out-of-plane provisions. I think it's 1.25.

MR. KLINGNER: We had it and made the conscious decision to take it out.

MR. TAWRESEY: Oh, we did?

MR. KLINGNER: Yeah. And the argument for taking it out was that -- we are dealing with a load that is being applied to the steel at a response rate that is typical of a fundamental period of a shear wall building, okay, say two- or three-tenths of a second. And that what we ought to do is make sure -- to make sure that the gauge length is long enough, but it's a completely different situation for walls in-plane than it is for walls out-of-plane or beams.

MR. TAWRESEY: Okay.

MR. KLINGNER: This is certainly something that seismic can revisit.

MR. DILL: There is reference out there that says in concrete that kind of reinforcement levels -- that they tend to form a single crack at the base and then the reinforcement ruptures. I have looked at the reports. Of course I haven't looked at all of the specimens or anything like that. Because the one way to look at what we have done here is in practice what their code provision is. It says put .007 percent in the wall and then run checks to see if you are okay. In essence what we are driving people to do, and I'm wondering whether .007 percent is too low because you are down at say half the thermal steel for concrete walls. You are very low. This is very light reinforcement that I am talking about. You better make sure that you get ductility.

MR. KLINGNER: The problem that we have is that it leads to very small bar sizes, and a very small bar size gives you a small gauge length along the bar, which translates into a small rotation at fracture. The bigger the bar is, you start to yield the bar. The greater length along the bar, you will be cracking concrete or masonry or whatever you have. If you shrink the bar down to a tiny whisker, because the bond stress stays as a function of the perimeter of the bar, the ratio of perimeter to area gets greater and greater until a hairline crack causes the bar to fracture. That's the problem with bed joint reinforcement and ductility. A problem. So it drives us into using very small bars which tend to give us a single crack rather than a family of cracks up the wall.

MR. TAWRESEY: So we need minimum size bars.

MR. KLINGNER: No. What you do is you de-bond the bar, but it's an additional problem.

MR. TAWRESEY: We don't have to worry about that. That happens automatically.

MR. KLINGNER: Well, but you --

MR. TAWRESEY: You convince the rest of the world that that is okay.

MR. ABRAMS: Reinforced ungrouted masonry.

MR. WEIGEL: That's another wall type.

MR. HUSTON: I'm sorry to keep coming back to the same thing. Look at the buildings that we currently are building. Look at buildings that are out there that went through Northridge, and they have lots bigger bars than No. 4 bars. They have No. 7. They have No. 8 and No. 9's. And now we are going to say that we are limited to No. 4's.

MR. KLINGNER: And the thing is, both points of view are right. Okay. The reason why those big buildings with the No. 8 and No. 9 bars have done okay is because they were stronger than we gave them credit for. We didn't need a lot of inelastic behavior.

MR. HUSTON: But, those were designed with bigger R's than we are using now.

MR. KLINGNER: Sometimes.

MR. HUSTON: Many of them.

MR. TAWRESEY: Most of them.

MR. HUSTON: I think they were designed with R = 8 or 6.

MR. DILL: Lower base shears.

MR. HUSTON: What we are saying is for masonry as opposed to any other material we are going to now say I have to reduce my R and make the buildings ductile. We are going to design the building for more shear. And once again I think the answer for most designers is to switch to another material.

MR. TAWRESEY: If we take the example that we did here, I guess if I designed a six-story apartment house with 12-inch walls in the base, that's probably the last job I do for that architect.

MR. HUSTON: Or owner.

MR. TAWRESEY: Or owner. I think that is a true statement.

MR. KANONIK: I would agree with that. Some of the buildings we have done had contractors who want to go to smaller than a six-inches.

MR. BROWN: It wouldn't have been a lesser wall than ASD.

MR. TAWRESEY: Just down the street here there is a 20-story eight-inch block hotel. They are all over the place. In Nevada there is a 28 story --

MR. HUSTON: How long ago was that built, John?

MR. TAWRESEY: 20 years ago.

MR. HUSTON: So that used a K of 1.33 which is an R of 8.

MR. TAWRESEY: Put it in perspective, you go five stories, it's wood frame metal studs. This would be a six-inch concrete. Right?

MR. HUSTON: Yeah. Not if I designed it, because I don't build six-inch concrete walls, but you could.

MR. TAWRESEY: So the 12-inch wall for a six-story building is way off the scale relative to the existing building stock so much so that it is why I made the statement. If we designed this, that is probably the last time I worked for that architect.

MR. HUSTON: While it's better than what we currently have, it's still going to drive us to totally different types of seismic force levels that are double what we are currently designing, so in order to be able to design a masonry building, we are going to have to double the base shear. It's already higher. It's already higher than that of concrete. So we are going to have base shear for masonry buildings two to three times higher than some of the concrete buildings.

MR. KLINGNER: Well, there is two ways of addressing that. The first one is to keep pushing for a rational look at where R's come from. The second one is address the consequences, at a system level, of toe crushing of elements.

MR. WEIGEL: Do you think your former point can be done without including the materials?

MR. KLINGNER: Of course not.

MR. THOMPSON: That's been the big block up to this point. We can change the R values but they are all scaled from other materials.

MR. DILL: Why don't we establish our own ductility requirements?

MR. KLINGNER: Triple secret probation.

MR. TAWRESEY: Maybe that is a subject of another meeting.

MR. KLINGNER: The answer for the masonry technical community is to lobby harder within the BSSC-PUC and ASCE 7.

MR. TAWRESEY: Maybe that's the subject of another meeting where we bring some people that have the horse power to put something together so that the masonry industry can be more influential.

MR. KLINGNER: We have done that. We are doing that now for autoclave rated concrete to actually get a logical rational procedure for R. But you get into that smoke-filled room where steel starts off with 12, you know how it works.

MR. TAWRESEY: So what kind of R is autoclave? Probably 6.

MR. BROWN: What does styrofoam get?

MR. TAWRESEY: Obviously very ductile.

MR. HUSTON: Just for a second. Let's go back. Because this is a good environment maybe to bring this up although we are at the end of our time. The building you talked about 20 years ago R was the same for every shear wall building.

MR. TAWRESEY: Right.

MR. HUSTON: It was a system. It was a boxed system. Since then we have had the changed to R_{cw}. Then we had more Rs, then we had changed to the R and we have a proliferation of them. And at every step of the way masonry has lost ground.

MR. KLINGNER: Yeah.

MR. HUSTON: Now we are talking about -- now we have got a Rho max provision that says we can't build it, and so we are saying to ourselves we will take R the wrong way to be able to solve the Rho max problem. I think the Rho max problem needs to be solved.

MR. KLINGNER: Do you think that the two things that I suggested first looking to approach of the Rho max problem with other materials within a rational way combined with looking at the system consequences of toe crushing, because the toe crushing is what is driving the Rho max?

MR. HUSTON: And yet FEMA, the FEMA document for repair of concrete and masonry buildings that came out of Northridge lists typical damage that you see, and there are all of these pictures of typical damage, and the first one for masonry is toe crush. It's expected. And yet so from the life safety code provision FEMA says this is the damage most like you find out there, and this is how you fix it. And here we are saying we want to restrict it.

MR. KLINGNER: That doesn't seem like a contradiction but it may mean -- it may mean that if we have multiple elements that the toe crushing isn't fatal.

MR. THOMPSON: Or some elements for an 80-foot wall and on either one I don't care. If a have a six-foot pier, I might get more concerned about it.

MR. TAWRESEY: Okay. We are out of time. What we will do is I use email to stay in contact with everybody. We will get the record and definitely the first step is to send the record. We will email it and we can change the whole -- but there will be one record, the original, which may or may not be published. But the intent will be to grab some of what happened here today so that it can be spread further outside this room, some of the key points, which I think it will be valuable.

MR. KLINGNER: I have a request. A lot of this is really sort of items for MSJC also items for TS 5.

MR. TAWRESEY: Right.

MR. KLINGNER: It would be real helpful if since most of us are in those groups we just put on different hats and go to different meetings. It would be real helpful if we could have early access. Is that okay with your sponsors?

MR. TAWRESEY: Oh, I'm sure it will be. We could take this and send it right away. The only thing we have to do is make sure that everybody agrees with what is down there first.

MR. KLINGNER: Oh, yeah.

MR. TAWRESEY: If it's okay with this group to go publish. And then we just will -- I think that was my intention.

MR. ABRAMS: I don't see any problem with that.

MR. TAWRESEY: We are going to get Western States credit for it. It would be nice to publish the trial designs. So if you could send me emails of your trial designs and some hardcopy.

MR. BROWN: Mine will have to be hardcopy.

MR. TAWRESEY: And then we will stick that in as appendix as the background to this.

MR. HUSTON: With or without names.

MR. TAWRESEY: I don't need the names.

MR. JOHNSON: I really think we need to concentrate on what Ed said and what you said. We have designed a 12-inch wall which would be the last job you would ever do for them. Yet I have eight-inch walls that survived Northridge that are higher. I mean, just put it this way. My bosses are smart people. They are not engineers, so they wouldn't understand this. All they would understand is, boy, that wouldn't make sense. In other words, what we are doing -- I mean, that's all they would hear. It doesn't make sense.

MR. HUSTON: Do people think that was useful?

MR. BROWN: Let me tell you why I think it was useful. I learned some useful things today. It is hard to wave a flexural wand. In the spirit of Rho max. Take that negative P and -- I've learned a lot today.
(Proceedings concluded at 3:59 p.m.)