Design of Connections in Steel Structures
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Lecture - 9
Fillet Welds - 2

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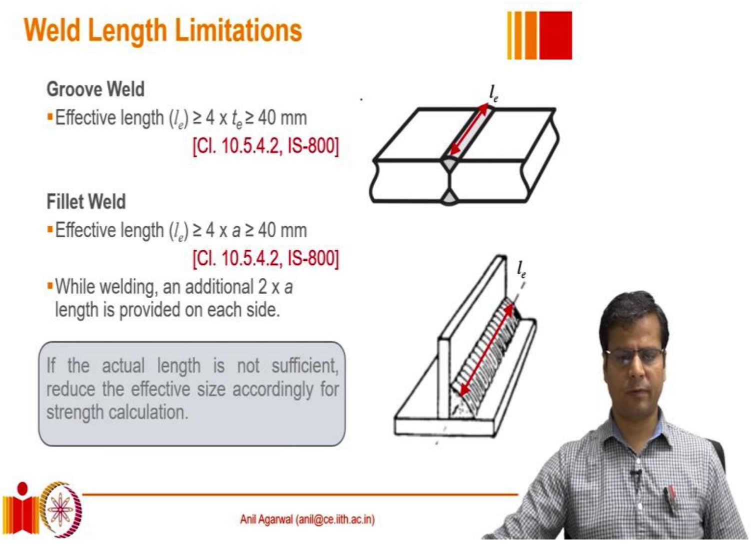

In addition to the size requirement for the fillet welds there are also some requirements for the length of the fillet weld, as well as the groove welds. So, first we will talk about the groove welds. So, if we do a groove weld the effective length should be at least four times the effective throat thickness of the groove weld, whether it is a full penetration CJP complete joint penetration or a partial joint penetration type of a weld.

In the Indian code IS800 prescribes that requirement in clause number 10.5.4.2 also, this dimension should not be less than 40 millimeters. So, at least 40 millimeter weld should be provided at a time. Any weld that is less than 40 millimeters is not qualified as a proper structural weld it is only a tack weld. If it is a fillet weld again, a similar requirement is there if, in such a case, in a fillet weld instead of using the throat thickness which was actually the equivalent of the throat thickness of the groove weld.

The requirement is that the weld length should be at least four times the weld size. So, weld size is ‘a’, which is this dimension four times at least that should be the length of the fillet. In both these cases however ther,e may be a situation where the weld of length less than 4 times the weld size is provided if that is the case for design calculations, we should consider that length as the effective length of whatever is provided and reduce the size of the weld in such a way for our design calculations. Reduce the size in such a way so, which is the size of the weld is one-fourth of the length of the weld. So, for example if we are providing an 8 millimeter weld if we are providing an eight millimeter weld but the length of the weld we have provided is only let us say 24 millimeters. So, this does not satisfy the requirement of length effective length should be greater than 4 times the weld size then what do we do?

We take this as the effective length and then the 'a' effective will be reduced to 24 divided by 4 which is 6 millimeters. So, for our design calculations we will not use the actual size of the weld we will use only six millimeters. So, we will reduce the weld size. Also, one must notice that whatever weld we provide as per the design and the drawings the welder at the site will also add some length to that weld and an additional at least two times the size of the weld is usually added to the fillet weld on each side.

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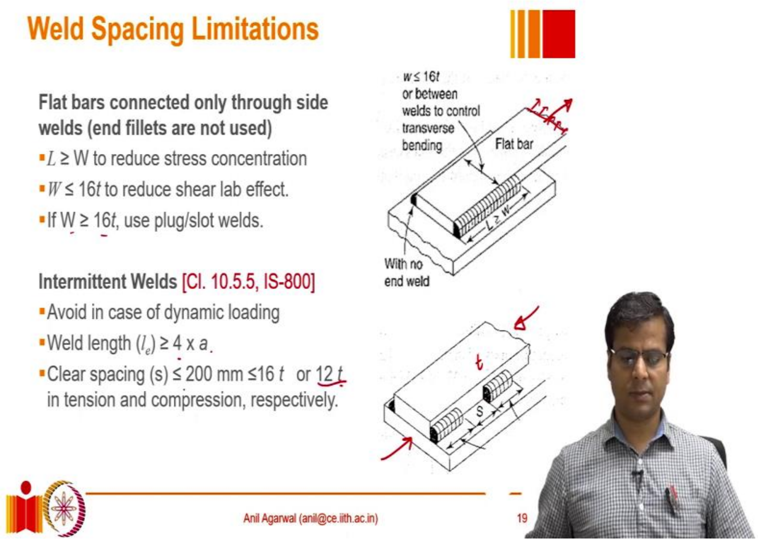

There are also some requirements about the placing of the welds in terms of how far apart the welds can be in the direction of the load, in the direction perpendicular to the load and so on. So, here is an example of two plates that are being fillet welded together through a lap joint. So, this is a flat plate it does not have any angle or anything any protruding part outside and if it is subjected to a tension force demand there are a couple of requirements for it.

One is that the length of the weld should be at least equal to the width of the plate that is width of the plate is actually governing the spacing of the two fillets which are resisting the load. This is basically required to reduce the stress concentration if this weld is very small in comparison to the width, the edges of this weld will be under very high stress therefore at least a certain length of the weld is provided. So, that those stress concentration at the edges can be minimized in addition to that the width of such a plate which is being resisted through fillet welds only at the ends the width of this plate should not be greater than 16 times the thickness of the plate, what happens if the plate is thick thicker wider than the 16 times the thickness of the plate in that case when the load is being applied there is a very significant shear lag effect the load is acting everywhere.

So, it is away from the weld it is uniformly distributed but as we come closer to the welds most of the force is getting transferred from the edges because that is where the weld is and there will be very large shear lag effect in a way that will reduce the strength of the the connection too much. So, in order to minimize that shear lag effect we should make sure that this width w is not more than 16 times the thickness of the plate is less than 16 times the thickness of the plate.

If however if this width is more than 16 times the thickness of the plate we have to make sure that this plates are not only welded at the edges but it is also welded at the end or also we may use plug or slot welds along the length of the along the length and width of the plate and join the two plates together through those plugs that will allow us to distribute the force more evenly between the plates through welds.

If we are going to use intermittent welds which are often required because the load requirement is sometimes very low and we do not have to cover the entire length of the weld using full size weld container slip weld. So, we may have to often go for intermittent welds however we should avoid those intermittent welds in case of dynamic loading conditions why because we as we discussed the edges of the ends of any stretch of a weld are the areas where high stress concentration takes place.

So, if we provide intermittent welds that means we provide welds in smaller sections several small belts which will produce several areas of high stress concentration and we should try to avoid it. So, especially if the structure is subjected to dynamic type of loading conditions where there is a possibility of fatigue we should avoid intermittent welds. Again the individual welds should be of at least four times the weld size length as we have discussed before for a regular weld.

In addition to that the clear spacing between two wells should be not more than 200 mm or should not be more than 16 times the plate thickness or 12 times the plate thickness depending on whether the welds are in tension or compression. So, if the welds are in tension that is if the members are subjected to tension they are being pulled apart in this case the ‘S’ value should not be more than 16 times the plate thickness this plate thickness and should not be more than 200 mm both conditions have to be satisfied.

If the weld is subjected to compression then this s value cannot exceed 12 times the plate thickness or 200 mm whichever is less.

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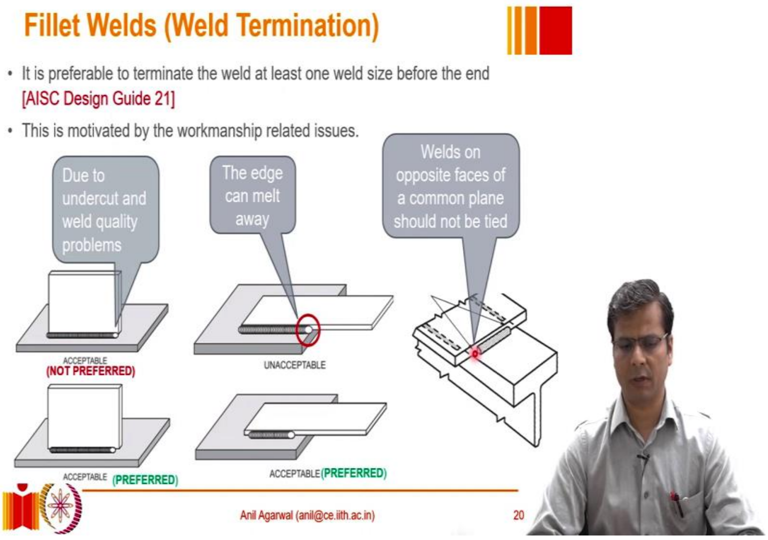

As a designer we should be mindful of this fact that terminating a weld can be a critical factor in the overall performance of a structure. So, generally it is not preferable to terminate a weld that is the end of a weld being located at a position where there is already a perpendicular edge. So, for example between these two options here you can see there is a horizontal plate in both cases and there is a vertical plate which is being fillet welded with each other.

Now it is general understanding that the second option is a preferable one even though the first one is also acceptable but specialty care needs to be taken and therefore it is not really preferred option. If one wants to continue the weld all the way to the edge it is better to continue the weld all the way around and that might that might be a better option. Why do not we continue the weld all the way to the edge and stop? Because then the quality of weld right at this location kind of becomes very difficult to assess and there is a very high probability that the quality of weld would not be good there will be some kind of an undercut some of this parent material would melt and leave a sharp corner there. So, therefore it is generally preferred that we stop the weld at least one weld size away from the edge, and that is how we should design the welds.

So, if we require the weld to be slightly shorter, that means we may have to increase the weld size to compensate for that reduction but we should prefer not to come all the way to the edge and stop. If we have to come to the edge, we should it is much better to just continue all the way around. A similar example is shown here again. So, this type of situation in fact is if one of the plates is extending beyond the other plate.

In such a situation, it is not even acceptable to stop the weld at this edge it is in, fact mandatory to stop the weld slightly before that edge because again at that edge there is a possibility that edge may melt away and will reduce the overall cross section and also it can lead to unwanted sharp corners in that location. Similarly like these also there can be a possibility where we have to weld or provide fillet welds in the opposite phases of a common plane.

So, for example here you may see these two welds are basically between this flange and this plate under the plate and this weld is again between this flange of the beam and this plate but this is above the flange. So, these two welds are on opposite sides of the plate and in such a situation it is better to not make them continuous it is better to terminate them separately again the risk here is because of this chain certain change in direction of the weld.

Where it has to turn in two planes simultaneously there is a very high likelihood that a large portion of this plate will be melted away and we will be left with a big cross section there that is where the failure may begin. So, mostly these requirements termination of the weld are governed by the workmanship related issues.

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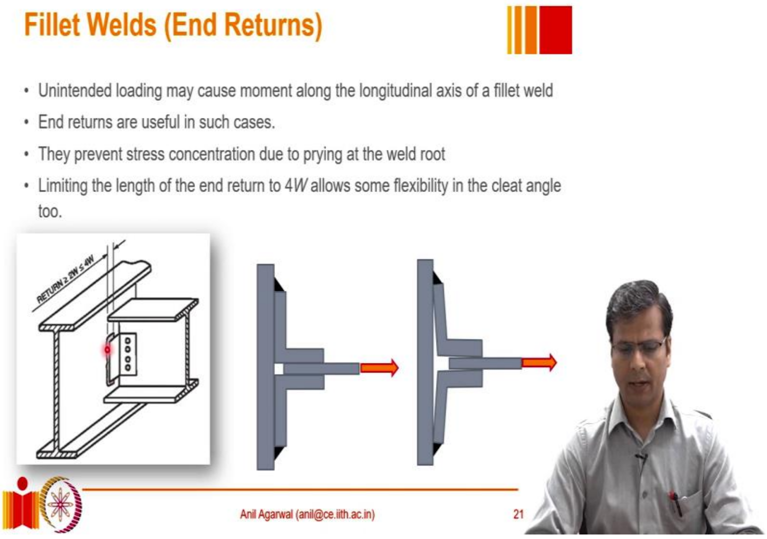

There are however some situations where we may actually want to not terminate the weld right before the end but continue the weld slightly further and then terminate it such a situation is called an end return and an end return. This end return type of a weld is particularly useful in such scenarios here I am showing you through an example. Let us say the same weld that in this connection you may see there is a beam which is connected to a girder and this connection is through a cleat angle.

So, there is a single angle here maybe there can be a double angle connection also the two connections behave very in a very similar fashion. So, if let us say there are double angle connections. So, there is a double plate connection. So, both angles are welded to the girder web and they are bolted to the beam web. The weld between the angle and the girder well are shown here as this fillet welds this is this portion of the fillet weld that is shown in the triangle here right.

Now generally such a connection is designed for a vertical load right that is a shear load primarily and if shear load is the only load acting on this weld this well even if it is not returned at the top and bottom it would perform perfectly fine there is no challenge there is no issue. However in such a scenario you might realize that this connection behaves almost like a simple connection.

And if it is a simple connection this at this end the beam is likely to rotate because at simple connections the beam ends can rotate they cannot resist moment. And if it is going to rotate this is going to lead into some kind of a pull force acting at the top edge of this angle which is shown here. So, if you look at the top edge of this angle from the top looking from the top it this angle at the top will be bending or deforming like this.

In such a scenario this particular fillet you might notice has a moment acting in its longitudinal axis which is basically causing it to pry open right at the root which we have as we have discussed before is not a desirable way of loading of this connection. Now this type of loading that is happening in this one is not really the intended type of loading because this weld is primarily designed to resist the vertical load only.

However there can sometimes be some unintended type of loading which can introduce these stresses and these stresses could be very bad for the health of this connection. So, in order to prevent this kind of a stress concentration it is advisable to not terminate the weld before the edge or before the corner but it is better to go around the corner slightly and go at least by a distance which is at least two times the weld size or less than four times the weld size.

So, there is a such recommendation available in the international codes which recommend us to go at least this much of the depth inside particularly on the side where there is a possibility of prying action therefore often this type of a weld is provided only on the top and may not be provided at the bottom because in kind of a in a simply supported beam we are likely to have tension at the end or a prying opening of at the top and that is where this has to be continued at the top okay.

Why do not we weld the entire length of the entire width of the angle at the top because also we want this connection to continue to behave like a simple connection. So, if we weld the entire width of the angle that will effectively restrain the movement completely and would again introduce areas of sharp stress concentration. Therefore we will only up to a certain width which is considered as four times the weld size and at that point we stop.

So, that the rest of the angle. So, let us say the weld is this much of width of angle is already welded the rest of the angle is still free to deform and as a result produce some level of flexibility in the connection.

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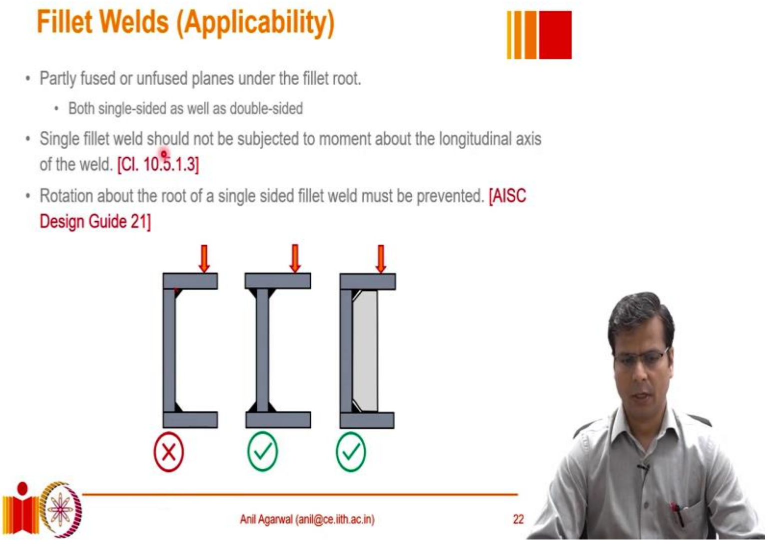

As we just discussed the fillet welds behave somewhat similar fashion as the partial joint penetration welds. So, fillet welds since they provide a welding between two perpendicular surfaces and there is always a portion of the combined surface which is not fused together and whether the welding is done on one side or whether it is done on the two sides in both cases there will always be some portion of the surface which will not be fused together and that is where the cracks can start.

So, we have to be mindful of the situations where the crack can begin here and we have to make sure that such type of learning situation should not arise. So, here through an example we can understand if we make a built up section which looks like this and if there is a possibility of a load acting like this on this flange, we might realize that this will again lead to an opening of the root in this weld which is not a desirable outcome right.

Because that is where the crack can very easily start however this can be prevented easily by one of the two ways either we can provide stiffness here. So, if we provide a stiffness here this stiffness can carry the load directly and transfer it if the stiffners are welded to the flanges these stiffeners can transfer the load to the bottom flange without having to rely on the or without having to transfer bending moment to this weld.

Alternatively we can provide double sided welds. So, double sided welds do offer some level of rigidity. So, if the moment is applied here there is a decent amount of level available. Now in this case in comparison to what was available here and as a result the stress concentration at the root is minimized. So, this recommendation is available in our Indian IS800 code also which specifically states that single fillet weld should not be subjected to moment about the longitudinal axis of the weld.

And the same guideline is specified in the American design guide also which specifies that rotation about the root of a single sided fillet weld must be prevented.

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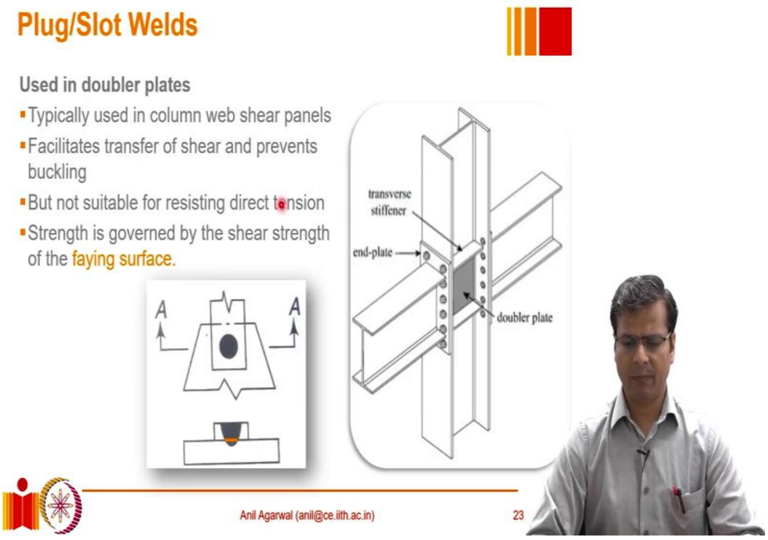

We will briefly discuss the issue of plug and slot welds we had also described it in the beginning a little bit. So, basically what happens is that plug and slot welds are not really a very frequently used type of welded connections they are actually a variation of fillet welds. So, in a fillet well we typically provide a weld along an edge in a plug or a slot weld. So, what you see here is a picture of a plug weld. Where you have two plates this could be the cassette plate and this is another bar which is to be welded to the gazer plate we drill a hole in the bar only and then we place it and then that hole is filled with the filler material building material. And as a result what you see here in orange color this small line is basically the new root of this weld and that is where the failure should be happening and that is that will control the design strength of this weld.

So, such welds are not typically provided for resisting regular tension or bending moment situations however they are very useful in situations such as the one shown here. So, for example in shear zone shear-dominated zones which is typically seen in a shear panel section of a column that is subjected to large bending moment demands under earthquake conditions for example.

In such a situation the shear panel needs to be stiffened. One option to stiffen that shear panel is through providing a double plate this doubler plate is expected to deform along with the weld of this column. If we weld this developer plate only on the outside four edges there is a possibility that this plate will buckle independent of the weld and therefore the total overall stiffness of the panel will not increase sufficiently.

One option to prevent that is actually to drill holes in this doubler plate at a few locations and then weld and provide this kind of a plug where it is to combine the two plates together. So, through these types of plug welds we can make the two plates behave simultaneously deform simultaneously. So, in such a situation shear dominated situations they are more frequently used.