e-conference on Indian Seismic Codes
(January 26 - February 8, 2002)
hosted by
National Information Centre of Earthquake Engineering
Indian Institute of Technology Kanpur

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Lapping of Vertical reinforcement in Column

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Hiren G.Desai [Saturday, January 26, 2002 8:39 AM]

Respected Sir/Medam,

First of all congratulation for hosting E-conference on this very important subject.
As per I.S. 13920 Vertical bars of Columns shall be lapped in regular zone (Mid-height) only and it should be staggered i.e. not more than 50% of bar can be lapped at one section. In addition to that minimum clear vertical distance between lapped bars shall be 0.25 times lap

length. This is mant times not possible for the multistorey prkject having floor height of 2.85 meter (9'-6") with bar diameter above 16 mm. Are same provision prevails in other codes? In many text books any drawings of very reputed structural firm of USA & UK, they are showing all column bars laped at mid heigh but at one section only. I request you to through some light on it as well as any practical solution available.

Thanking you,
Hiren G.Desai

Dipak Shah [Saturday, January 26, 2002 10:32 AM]

Dear Sir/Madam
I fully agreed with Mr.Desais' quarry.
There is no other practical solution for 3mtr. floor ht.We have detailed [& executed] 50% splicing criteria by providing varying length of reinforcement [Mid ht. to Mid ht.] Major problem is in reduction of size of floor to floor column[depth] when we spliced the bars at mid ht.Further hoop[135 degree]stirrus makes compaction difficult[needle conflict with stirrups] & to overcome provided 180 degree seissmic hoop. For Ductile detailing its a great responsibility on us to detail it,inter-act with site supervise & Bar bender so that provisions will not remain on paper.Bar bender is the key person and he should get remuneration for his exhaustive work.In my view,it will take long run to change our attitude and practice for Ductile structural system [For non ductile system just lower value of Response reduction factor " R " as per IS 1893-2001 Tab.7 is not suffice for the safety of structure]

Dipak Shah

Alpa Sheth [Saturday, January 26, 2002 10:51 AM]

Hello Hiren,
Welcome!
Yes, column lap splices as per IS 13920 is a problem that haunts most designers!!! I share your difficulty. The ACI-318M-95 (Cl 21.4.3.2) also mentions that the lap splices shall be permitted only within center half of the member length and shall be proportioned as tension splices. It is silent on the max 50% requirement of bars being lapped; however in CLause 12.17.2.2 and .3 it states that if the bar stress exceeds 0.5fy in tension or if more than 50% of the bars are lapped at one point, the lap splices shall be Class B type (Splice length=1.3 Ld). UBC (CL 19.21.4.3.2) is also silent about the 50% requirement and has the same requirement of 1.3 Ld splice length (Clause 19.12.17.2) for similar conditions. One way one could try to conform to the code is by using two floor height column reinforcement bars. EVen then, it is not always easy to follow all three conditions- 50% bars only, mid height splicing and min clear distance between laps = 0.30 lap length for 9'6" floor hts., especially for larger diameter bars.

If all three are not being satisfied I would follow the first two and give least priority to the min clear distance between laps =0.3 lap length (IS 456 clause 26.2.5) as long as you are giving special confining reinforcement in the lap region. I think this will then conform to the clauses of IS 13920 and IS 456. I'd like to hear from other designers how they're solving the problem.

Regards,
Alpa

Narendra Prabhakar Walavalkar [Saturday, January 26, 2002 2:13 PM]

Dear Sir / Madam

I would like to add one point
As per ACI, since generally all of the column bars will be spiced at the same location, a class C spice will be required. Required length of the spice is 1.7 x Ld Hand book by Mark Fintel discussed the same thing in relevent chapter.

Thanx
Narendra

Anal Shah [Saturday, January 26, 2002 4:47 PM]

The lapping clause is more problemetic if someone wants to add one future floor at later date which happens in India quite commonly!!!

Anal Shah

Alpa Sheth [Saturday, 26 Jan 2002 5:10 PM]

Hello Narendra,

I'm not sure if there is a Class C splice reqmt for column reinforcement splicing in the revised ACI code -from ACI 318M-95 code and later revisions. You are right, the earlier ACI code had a Class C splice too!
Regds,
Alpa

Suhas Mujumdar [Saturday, January 26, 2002 5:26 PM]

Congratulations for hosting this conference ;this is unique and one of its own kind.!!
Since morning i was attending the discussion regarding column splice. I have heard that in Bhuj area Authority is restricting the height of the structure at present . for a common people if the house with foundation for two storey is built at present what will be in future if the authority will allow further height . at the same time the quastion of column lap at top story will arise.

suhas mujumdar

Jitendra K. Bothara [Sunday, January 27, 2002 9:18 AM]

Hi Colleagues,
It is interesting to join you all on this issues. I have few remarks (?) on lapping of bars, seismic zoning map and other issues. These are:

1. Lapping of Column Longitudinal bars:
Regarding lapping of longitudinal bars of column, NZS3101-1995 (New Zealand Concrete Structure Standard), Cl. 7.5.1 permits use of full strength welding (butt or lapping) at any location (including potential plastic hinze (PPHz) location). Of course, it would also depend on quality of steel and quality of welding (which could be difficult to achieve in large majority of projects in India or Nepal). Further, the same clause elaborates, other type of splices (normal lapping) can be provided in any location where PPHz can be eliminated. This can be achieved in columns by adopting Capacity design approach for flexural bars (IS13920 covers capacity design of transverse bars but not flexural (if I remember well!)) there by compelling plastic hinges in beams (beam-sway mechanism). Of course even then column bottoms will develop plastic hinges in severe earthquake. This scenario makes things easier. Now, in case of the bottom storey, lapping can be avoided or only half of the bars can be lapped there and rest can be lapped in upper storey. In case of upper storeys, bars can be lapped in any location, of course, tied with special transverse reinforcement. NZS3101-1995 (Part 2: Commentary) explicitly permits lapping immediately above floor level in upper storeys.

Of course, if capacity design approach for longitudinal bas is not adopted, the best thing is to avoid PPHz locations in column/ beams for lapping and using full development length with some margin (always for tension). The issue of staggering of lapping should be of least priority if storey height does not permit it.
But if we do not follow beam-sway mechanism, column-sway mechanism could lead the building to total unstability during severe earthquake.
…..
It is all for now. See you later.

Jitendra K. Bothara

M. Hariharan [Sunday, January 27, 2002 9:51 PM]

I saw a catalogue of a threaded coupling for reinforcement which eliminates the need for lapping. Has anybody tried it? I have found it conceptually advantageous for marine piling, where, from practical considerations, nearly 40% extra steel is required due to this 50% and lapping requirement.

M. Hariharan

Arvind [Sunday, January 27, 2002 10:12 PM]

Dear Participants,
I would like to share my thoughts:
2. Splicing of reinforcement for columns:
It is very important that we avoid the splicing of reinforcement in the base story (which accounts for maximum shear during earth quake). This can be done in majority of cases where foundations are not very deep (less than 4 meters) and one can take advantage of 12m length of bar.
Cheers! Keep it up.

with warm regards.........
Arvind

Moderators [Monday, January 28, 2002 1:12 AM]

Dear Colleagues,

We thought we'd summarise the key technical queries that have been discussed
in the past couple of days.

1.0 Lapping of Vertical Reinforcement:
This was one of the first issues we've discussed. From the posts it appeared that there was a general consensus that trying to implement the following three requirements of Column Bar splicing was sometimes impracticable:

a) max 50% bars to be spliced at a section,
b) splicing at in the middle half of the column height, as well as
c) trying to keep a clear distance between laps of 0.3 development length

Reference to international codes brought home the point that while all codes frown on more than 50% of bars being lapped at one location, they allow it after a punitive increase in the required development length. This is significant in situations where you are tying the column reinforcement cage at the ground level, complete with column rings and lifting this cage by means of mechanised methods such as a Crane. (This is not an uncommon practice for towers). In such cases, bars of unequal length become very difficult to maneuver and it may become imperative to have a system by which you may splice bars at one location without compromising on the structural integrity. Presently we do not have such an option available to us in IS 13920. This could be reviewed in the next proposed revision of this code. We would welcome more discussion regarding this and would like to hear apprehensions of changing this clause 7.2.1 in IS 13920.

There were issues raised about what does one do or those buildings where there is a future floor provision. Would the lapping of bars be all at the same location? It appears in such a case you would need to provide taller pedestals for the future columns, up to say about half the proposed floor height.
….
Regards,
Alpa Sheth and Durgesh Rai

Srinewas [Monday, January 28, 2002 2:37 PM]

Dear Hariharan,

I would request you to send me the details of the threaded coupling for extension of reinforcement. Recently we have added one floor on top of the ground floor of Lab building at Mathura.We faced the similar situation of lapping of reinforcement.Finally we devised the following procedure:- 1.Where ever the existing bars were available welding was done for extension of reinf.

2. In other places the Bars to be extended were chemically grouted into the existing columns, by drilling the existing column top and pouring the chemical into the hole and then immediately putting the column bars to be extended into the hole.Within 10-15 minutes the anchorage of the bars grouted was very high, sufficient to take the entire load of the structure above. How much it would help against EQ is not known to me. If somebody can enlighten me on the subject, I would appreciate that.

SRINEWAS

Rajiv Sharma [Monday, January 28, 2002 6:16 PM]

Hello Friends:
This is in response to issues raised on lapping of vertical bars in columns. Although it is not easy to implement requirements of cl. 7.2.1 of IS:13920 but it largely depends on the willingness of people involved in design and construction. The method of lapping bars in alternate storeys and lapping them in middle height of column works reasonably well in most of the situations. The difficulty before a site engineer is to find out, which bars are to be lapped at a particular location. We have seen that many a time designers make a sketch similar to figure 9 of IS:13920 in their drawings and leave it to the site engineer to figure out lapping arrangement. This is not correct. I feel that the problem can be tackled by designating column bars in 2 groups. Only one group's bars can be lapped in a storey while other group's bars will be lapped in the next. If this information is given to the site engineer the confusion regarding lapping arrangement will vanish. I must admit here that working out column bars with this requirement will take considerably longer time for the design engineer. But it is worth that. Although double height bars do create problems in handling but they are not impossible. We have used this method on a number of projects very well and our experience is that it depends largely on the willingness of people involved.

Mechanical connectors like steel bar couplers can also be used but are in general expensive and hence we hardly see them on any of the sites. Other connections like welding are seldom used and the E.Q. codes are in general silent on the use of welding.
Bye for now

Rajiv Sharma

Arvind Jaiswal [Monday, January 28, 2002 8:04 PM]

This is in response to Mr. Rajeev Sharma regarding Lapping of Column Bars.

1. I am in agreement with Rajeev.

2. It is certainly dependent on the Designer how much time he is willing to spend on his design concept and how much he exerts control and spends time on the site (at least first time he has to).

3. This is also dependent on the type of fees he is charging.

4. In case it is run of the mill job, then he will give least details. (This may stop after Engineers Bill is enacted because after that Consulting Engineer will be held legally responsible and he will have to prove his proficiency in particular field he is practising or claiming to have expertise. At present Engineering Council of India is being formed and the meeting is slated to be held sometime on 5th or 6th of Feb) Of course this is off the subject of EQ we are discussing now!

5. In case Designer defines the playing rules in the beginning of the project, it should not be difficult even to avoid lapping the bars in ground storey normal buildings. Imagin a normal building having each storey height of 3m. Assuming general depth of foundation as 1.6 m to 3.0 meters and each storey height of 3 meters should take you to the first floor 50 % lap and then second floor 50 % lap with average bar length of 9m to 11 meters.

6. The problem faced will be erecting the already fabricated column as pointed out by the moderators. But the key is not to fabricate the column in the yard! The fabrication has to be done above foundation mat with the help of acrow supports by keeping L shape rings at 3-4 places vertically and then inserting bar by bar and enclosing the same with rings / stirrups after all the column bars are in position. It is certainly successfully tried detail on many sites. with warm regards.........

Arvind

Jayant Sheth [Monday, January 28, 2002 10:21 PM]

Dear friends,

In practice without consulting the engineer who is responsible for idealising the structural behaviour of the building, many additions, alterations are done by Users of the building thereby causing great danger to the structure. At times the fact is that the manufacturer of construction chemicals and adhesive claims very tall only with the intention of selling the product more. Out of anxiety at site some commit mistakes and use them without understanding the implication. This email falls under similar situation. Mr. Sriniwas has either grouted the bars with epoxy or high strength grout and also without making a valid test at site for performance adjudged the bar has been firmly fixed so that it will transfer the

force for the purpose it was kept. Such thing has to be stopped. Another alternative which has been employed by him is welding. The same could be better alternative than grouting. However, a proper welding procedure has to be adopted with appropriate welding rod. The reinfocement bar if is CTD than a careful evaluation of welding procedure is must. I think all htese details are available with experienced engineer. The splice bars has to be threaded than only it works better. It is not possible to thread the bars of dowels.
With regards

JAYANT

Anand Bhagwatwar [Tuesday, January 29, 2002 9:58 AM]

Dear Sir,

Why do the bars have to be lapped at mid height? This is the area with the minimum frame moments. Can the laps not be placed at the junction of beam and column? With additional lapping length? The consultant may make a number of recommendations in his drawings, but these recommendations are not always followed, especially in such matters where some prevalant practices exist. Builders or contractors who regularly interact with the consultant accept the changes recommended by the consultants. However this is not the case in many projects.

As the eq. codes are under revesion, this point can be discussed and suitable recommendations can be made.

Regards
Anand Bhagwatwar

M. Hariharan [Tuesday, January 29, 2002 8:22 PM]

SRINEWAS,

The brochure is with a colleague. I shall furnish details as soon as I get it (in a day or so).
Hariharan

Nilesh H. Shah [Tuesday, January 29, 2002 12:07 PM]

Dear Friends,

First Few days of the e-conference had frequent discussion on column bar splicing that confirms to provisions of 13920, i.e. max. 50 % bars to be lapped, mid-height location and staggered distance of 1.3*Ld. It is possible to accommodate all three provisions by using welded laps, mechanical couplers and alternate floor lapping. I share my views on the subject as follows:

1. In few of our recent projects, we have used welded laps. Welding reduces development length to a considerable extent. Due care needs to be exercised during execution to monitor quality of welds, especially because it is in vertical position. Welding rod, material, Instrument and procedure for welding shall confirm to relevant IS code of practice. Such work needs to be executed by skilled and qualified welder. Through testing, it is necessary to ensure adequate strength of weld and to establish that failure of reinforcement precedes that of weld. If required simple NDT of weld on site can be performed to verify its integrity. In our opinion for large diameter bars (>20 mm), this does not significantly add to cost, as it is partly compensated by saving in lap length. However, the procedure consumes more time, when compared to routine practice.

2. Use of mechanical connectors should also help in satisfying all three provisions. Information about such devices can be had from www.ishitaonline.com. This procedure is also time consuming as threading of reinforcement is required. Also, it appears costly. Moreover, we still await publication of formal IS code of practice for mechanical coupling of reinforcement.

3. Half the reinforcing bars being lapped at alternate floor level seems very convenient and easy option. In many recent designs we have implemented this. Of course, one has to overcome initial resistance and reluctance of the constructor. Routine practice of preparing reinforcement cage at ground level and lifting it to higher floors does not work here. It is almost certain that, reinforcement for column (and beam too) needs to be prepared "in-lay-position" at respective floors for proper ductile detailing. It is not difficult to maintain cage of 20 mm or higher diameter bars for a double floor height. Cage of smaller diameter bars would require some extra efforts to keep them in position.

4. Satisfying all three provisions of 13920 is not VERY difficult as being projected in then discussion. It requires more efforts on the part of designer and constructor. It may also require change in routine practice of design, detailing and execution. More elaborate drawings clearly indicating position of reinforcement laps for each column group is essential for easy communication to the constructor. Execution of reinforcement work too requires skill and expertise as that of structural steel fabrication. Due recognition to this aspect together with extra effort for elaborate detailing/drawings and "CAN DO" attitude of designer/constructor shall definitely help in smooth execution of work, while satisfying all three provisions of column bar splicing given in 13920.

Views from participants and moderators are welcome.

Nilesh H. Shah

Pawan R. Gupta [Tuesday, January 29, 2002 11:41 PM]

I have been following the discussions for the last couple of days and it is very interesting. Here is my 2 cents worth in response to the email by Mr. Srinewas and Mr. Hariharan. We in Canada sometimes use the Lenton couplers when we need to

1. Either leave the option for a structure to be extended
2. Have a very congested pattern of reinforcement i.e. more than 4%.
3. Have very large reinforcing bars eg. 45mm dia where the development length becomes very large.

From all of the research that I have seen they behave very well. The only caution is that these lenton couplers tend to be quite expensive. Some shop work is also required on the rebars to provide the tapered thread on the bar. If anyone is interested in getting more information you can to their website.
http://www.erico.com/erico_public/division/ConcreteReinforcing.asp#asia

Hopefully this is useful for the discussion
Best Regards
Pawan R. Gupta

Vijay Patil [Wednesday, January 30, 2002 3:58 PM]

DEAR FRIENDS AND FOES PLEASE READ THIS AND REPLY
We as engineers have to carry out brainstorming and try to find out solutions to our problems some solutions might sound stupid but it may work. I am trying to give a stupid (till proved to be brilliant) solution for the columns bar lapping problem and some academician or researcher has to find out whether it is stupid or good (and let me know too)
STUPID SOLUTION -1 starts here

Lets start form the very basics. Columns are designed to carry certain amount of Axial force and Certain amount of Moment. Our codes have given a lot of interaction diagrams to design columns with axial load and moments (even if they are in both directions).
Assume a square column having reinforcement along the peripherry. Designed of Axial force and the Moment M this is the normal design of column. The stupid design solution would be design the column with a group of bundled bars at centre of the column to carry most of the axial force. And the peripherry bars would carry part Axial and total Moment along with the concrete. I KNOW the idea sounds stupid but it may work. Only thing is that we have to change out interaction diagrams and design methodology of columns. The buckling of the core reinforcement may impose additional Moment on the column which needs to be handled.

How would that help??
-- Well you have a larger space to accomodate your reinforcement and dont have to work in the same limited space to lap your reinforcement.
--- Since the central core reinforcement would be quite less in number but tied together with ties would be rigid enough to stand upright two floors (may be with minor supports).
--- you can even weld the central core bars for direct transfer of axial forces which will not transfer much of stress to concrete (that is what actually bothers us when we lap). In this case you will have to weld less number of bars.
--- This does not mean that the concrete is not carrying any Axial load at all It will carry less load and hence the clause of lapping of columns bars in the peripherry could be relaxed.
Kindly express your views.

Vijay Patil

S.R. Satish Kumar [Wednesday, January 30, 2002 5:35 PM]

Dear Mr. Patil,
As you would have expected, your solution is slightly stupid but very good ! First the stupid part.

What you have not realised is that when you apply a bending moment to a column, it BENDS. It is no more straight which means that the centreline of the column is curved and any axial load will produce what is known as the P-delta effect (additional bending moment). You seem to be expecting such extra moment, but it comes not because of the buckling of core reinf but because of the bending of the column as a whole. Also to prevent the buckling of the core reinforcement, it will have to be supported laterally at frequent intervals. Both these problems, as you have rightly pointed out, can be solved by providing addiotional reinforcement for bending. The second problem, is that column reinforcement in RC has two main functions other than carrying axial load. It is required to take the tensile stresses produced by the bending moment. It also has to confine the concrete thereby increasing the ductility and energy dissipation capacity of the section. This means substantial bending reinforcement will have to be provided and it will be simply waiting for the earthquake to occur making it an uneconomical solution. A good example of the application of the above idea is the concrete-encased steel column where we use a steel section at the core and encase it in concrete. This also protects the steel from fires but the concrete simply spalls off during an earthquake unless extra steel reinf is provided. The good part is that your idea has been developed in several ways and found to give wonderful results. One such example is the core-loaded sleeved brace where there is a core reinforcement of high strength steel which takes all the axial load. This is surrounded by another steel tube which is filled with grout and together they prevent the core from buckling. The system is patented by Tube Products of India, Chennai. Similar systems have been tried by several Japanese researchers also. In conclusion, I would like to remind you that several methods exist to improve the performance of structures during earthquakes but the underlying problem with all is that they cost money - something which makes people back off from using it. I am sure that as a professional, you will fully agree with the above statement.
Satish

Vijay Patil [Wednesday, January 30, 2002 8:48 PM]

Thanks Satish,
That reminds me what could be ductile detailing in case of composite structures. Is it given in any of the codes.

Vijay Patil

Jitendra K. Bothara [Friday, February 01, 2002 7:23 AM]

In response to vsnl
Lapping column bars in beam column joint region can not be considered good. IS13920 even prohibits even lapping of bars just above and below the beam (plastic hinge region) because these regions are expected to go under plastic hinging (as Indian Standards do not require strong column and weak beam construction). Once the plastic hinge develped, bars will loss the bond strength. Of course, column bars can be lapped just above floor level provided that we can gurantee that plastic hinges (PHz) do not develop there (only possible if strong column-weak beam construction) and these bars are provided with full confining stirrups. In this case also, bottom of columns may develop plastic hinges, so in ground floor lapping in potential plastic hinging region (just above and below the beams) is not permitted.

Cheers,
Jitendra K Bothara

V. Levtchitch [Friday, February 01, 2002 3:52 PM]

I find this conference very interesting and I am delighted to follow it. Unfortunately I could not open the Indian Code itself and I am confined to discussions only. At the moment I would like to express a strong reservation against the use of welded laps in order to reduce a development length of longitudinal reinforcing bars. We have an experimental evidence that under cyclic (i.e.seismic) loadings it is practically impossible to ensure a failure of reinforcement before that of welds. Fatigue fracture invariably originates at welds. The reduction of fatigue limit can be as large as 65%. The strain capacity of welds is much lower than that of reinforcement and this is a fundamental problem. Although our experiments were confined to beams, it is a general phenomenon which holds true for columns as well. Moreover, the required quality of welding at building sites cannot be achieved. For this reason in Japan the electric welding at building sites is not permitted. Also I am a bit surprised that the "strong column-weak beam" philosophy has not been incorporated into codes. Or maybe I have got a wrong impression? Large deformations-just short of collapse-can be purposely allowed and plastic hinges are to be promoted in beams which provide the first line defence mechanism. Performance of beams governs the behaviour of a building as a whole. Yours sincerely

Prof. V. Levtchitch

R.L. Nene [Saturday, February 02, 2002 5:38 PM]

Mr. Bothra,

I strongly advise the lapping of column bars at mid height instead of a number of measures sggested. Splicing of clumns bars at the midheight is very conveniently done in quite few jobs in Mumbai. Of course, the contractor may need training. This is clearly shown in ISSE book on Design of Reinforced Concrete Structures for Earthquake Resistance. This book is available at the following address.:
Indian Society of Structural Engineers, 24, Pandit House, S. K. Bole
Road, Dadar west,

Mumbai 400 028. Tel/Fax 422 4096, tel 4365240, E-mail: isse@vsnl.net

R. L. NENE

S.P. Srinivasan [Sunday, February 03, 2002 8:52 PM]

Hello everybody,

1. Lap joints for column bars seems to be an appropriate area for experimental research work.
2. Please comment on the following argument for lapping all bars at mid-height with normal laplength:

Consider middle half of a column. Maximum moment will occur at one of the quarter points of the column. If lower and upper end moments are equal, this maximum moment will be 50% of end moments. If one end moment is zero, the maximum moment within middle half will be 75% of the larger end moment. Hence in an actual case the maximum moment within middle half will be 50 to 75% of the larger end moment.

In order to ensure strong-column-weak-beam condition, we will provide a minimum excess capacity of 20% in columns. This means that the actual maximum moment within the middle half will be 42% to 63% of the provided moment capacity of the column. Take this value as 60% of moment capacity

Refer Chart 45 of SP16 (This chart applies for most practical cases of column design). For a 40% reduction in moment, the reduction in reinforcement varies from about 40% to 60%. The larger reduction of reinforcement occurs when Pu/(Fck b D) is low.

If we assume that the maximum stress in steel in the middle half is 60% of the design stress, then theoretically required lap length is 60%. If we provide 1.7 times this lap length, the lap length to be provided works out to 1.02 times the development length.. As the lapping point moves closer to the point of contraflexure, the requirement actually will reduce.

So, can we lap all column bars at mid-height adopting normal laplength?

Regards

S.P.Srinivasan

Hemant Vadalkar [Tuesday, February 05, 2002 1:36 PM]

Dear fellow Engineers,

Please come out with solutions to difficulties faced at site during placing of reinforcement based on your experience. I am giving my views :

1. **Columns lap at mid height :** It is difficult to have 50% lap at midheight of the column. The general residential building Floor height is 2.9m. The lap can not be placed just above the slab or it can not be in the beam column junction. The available space left is 2.9m - 0.6m floor beam = 2.3m. If we have 25mm bar, the lap required is 1250mm. So, it is difficult to stagger the lap in the available floor height. To achieve 50% lap, the only option left is to provide bar length of two storey height + Ld. In this situation, supporting of column cage is necessary and there will be difficulty in case of column size reduction or reinforcement reduction.

If the activity is planned in advance with detail column drawings in design office and insisted at site, it can be done. Mechanical couplers can be used to avoid conjestion of reinforcement.
…

I would like to know the comments and suggestions from the experts.

With thanks and regards.

Hemant Vadalkar