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Shear Walls in RC Frames

Girish Dhanwani [Saturday, January 26, 2002 5:35 PM]

Respected Sir/Madam,
I have a question.
As per the IS:13920, all the buildings coming up in ZONE-IV and ZONE-V, should follow the ductility requirement, as per all the norms mentioned in this code itself. I would like to know, whether this is applicable to RCC frame structures (with some Shear Wall), or it is also applicable to RCC buildings, which are constructed mainly of RCC Shear walls and with almost no columns. In my opinion, such building can be considered as a RIGID building, and should be designed for performance factor of 1.6. On the other hand, if, as per code, we try to provide special confining reinforcement in all these shear wall to make them DUCTILE, it results into anormous steel. I would like to have your as well as other consultants views, in this regard.
Thanking you,

Girish Dhanwani

Alpa Sheth [Saturday, January 26, 2002 6:40 PM]

Hi Girish,
I'm afraid that you have to design Shear Wall structures as per ductility requirements of IS 13920 for Zone III (beyond 5 storeys), IV and V. Ofcourse you may use K=1.0. As you probably know, the special confining reinforcement is required only in the boundary elements of shear walls and not in the entire length of the shear wall. Ofcourse if you do not wish to provide boundary elements, you would need to provide special confining reinforcement along the whole length of the wall as per CLause 7.4. In the rest of the wall, you may provide transverse reinforcement as per requirement subject to a min of 0.0025 of the gross area of the section. WHat has been your experience about providing boundary elements? ne aften finds it is more optimal to provide boundary elements. Anyone else care to share their views?
Cheers,

Alpa

Vijay K. Patil [Sunday, January 27, 2002 12:18 AM]

Dear Friends
All the while it appears that we have been discussing Concrete structures.
>From IS:1893 it is not clear as to what is ductile design for steel structures. Steel as a material itself is ductile but such a clause is not given in any of the codes. (niether in IS:4326 which is referred in IS:1893).
-If you are to take the performance factor 1.0 for steel structures does it make it mandatory to have Moment resisting frames. In steel we have Rigid, Semi-Rigid and Simple designs. According to the clause it means that if one adopts K=1.0 then it better be a Rigid Design irrespective you provide a shear wall (in Conc) or braced frame(in steel) for the building. In reply to a query by Mr.Girish on Ductile or Rigid it has been mentioned that for concrete structures in case you are designing shear walls as ductile then K can be taken as 1.0 however in normal practice in case shear walls are adopted it is assumed that the columns (frames) would carry only 25% of the force (saves a lot of money and headache of lapping I guess). This is not clear in the code. Well when we are assuming the floor to be rigid it is very obvious that the horz shear would be shared by the vertical members on the basis of its stiffness. In such a case why should there be different performance factors such as 1.0, 1.3 or 1.6 etc. (Unless ofcourse the building is unsymmetrical and is subjected to torsion). Dont we all know that a buiding with shear walls/ Bracings perfrom better than pure moment resisting frames then why the performance factor is more for buildings with shear walls/Bracings and less for Moment Resisting frames.(This question is more pertinant to Steel

Structures). As far as the new code IS:1893 (Draft) goes Square would be the order of the day Square Columns, Square Buildings etc. I guess it is enough typing for the day I will be waiting for the replies. Last but Not the least I must congratulate Dr. Sudhir K. Jain and the entire team for organising this e-conference on e-quake.

Vijay K. Patil

Arzhang Alimoradi [Sunday, January 27, 2002 2:17 AM]

Hi again;
Interesting discussion Vijay K. Patil, thanks. I am unfortunately not very familiar with your national building code, but just let me drop my two coins on: "... Dont we all know that a buiding with shear walls/ Bracings perfrom better than pure moment resisting frames... " Well. Not necessarily. And this's what makes our job as earthquake engineers amazing. It mainly depends on whole lot of things: How well the Shear walls/Beacing system are designed? High ductility / low ductility? Building height? Natural period? Site conditions? Even Tectonics of the region, in terms of expecting any near fault ground motion... I doubt if any general statement like that could be valid in structural design. But anyhow, interesting question. I appreciate if someone familiar w./ this issue in your national code can shed some light..., different steel structures and their response modification factors. Looking forward for your contribution.

Arzhang

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.
…
2.0 Design of Shear Wall Buildings:
A query was raised regarding the Performance factor K to be used for buildings which were essentially behaving as shear wall structures. It was discussed that as per present interpretation of IS 13920, in Zones III (above 5 storeys) , IV and V, the shear walls have to be designed as ductile shear walls (K may be taken as 1). However the special confining steel needs to be provided only for the boundary elements.
There are still some queries on this subject which have remained unanswered. These had to do with behavior of shear wall vs frame structures, justification of the differing values of K for frame and shear wall structures, behaviour of steel

structures and so on.
This subject is open to further discussion and in the days to come we hope participants will respond to the posted queries.
…
Regards,

Alpa Sheth and Durgesh Rai

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

Thanks Arzhang Alimoradi & R.P. Singh,
Sorry to reply so late as I was out of station. Yes I definately agree with you that the Structural Response depends a lot on various factors, as you rightly said it cannot be generalized. However it appears that if Shear walls or Bracings are provided (judiciously that is) they as the name suggest are good in carrying the building shear expected in EQ's.

Vijay Patil

K.N.Chandrashekaran [Wednesday, January 30, 2002 10:02 PM]

Can Cylindrical RCC walls going the full height of the building be considered as shear walls? Do only straight walls act as shear walls? Pl. clarify.

K.N.CHANDRASHEKARAN

Subhamoy Kar [Thursday, January 31, 2002 6:42 PM]

Hello...
This is in response to the E-mail from Mr. K.N.Chandrashekaran, which is included below.
Certainly cylindrical wall (up to full building height) can be considered as shear wall. If this wall is exterior wall its behaviour will be like a tubular structure. You may refer to the book "Tall buildings", by Taranath, wherein tubular structures are described. Probably it is a McGrow Hill Publication. If the wall is interior wall still it is supposed to have much higher stiffness than the total stiffness of building columns. Hence they will attract lion's share of the lateral load and thus acting as shear wall. Only thing is that floor beams and slab are to be properly tied with the wall for transfering the lateral load.
The formulae for calculating stresses in cylindrical shear wall will be similar to that of RCC chimney. In this context, the book titled "Tall Chimney", by S.N.Manohar (Tata McGrow Hill publication) may be referred to.

Thanks and regards.

SUBHAMOY KAR

Vasant S. Kelkar [Tuesday, February 05, 2002 10:07 AM]

February 2, 2002

With reference to the query of Mr. K.N. Chandrashekharan if cylindrical walls can be considered as shear walls, I may clarify the following:

Shear walls are basically vertical cantilever beams or beam-columns. To give some historical background: When underground nuclear tests were being conducted in Nevada deserts in USA in late 1940s/1950s, the engineers had to design the office/residential buildings in the neighbourhood complex to withstand large lateral loads due to ground motions caused by underground explosions. For good lateral load resistance they found RC walls to be very effective. Since these buildings were only one or two storeys in height, these walls had a small(less than one also) height to length ratio (span/depth ratio). Therefore, besides flexural deformations shear deformations contributed considerably to the total deformations of such walls under lateral loads and they had to be considered for analysis and design of these walls. Hence, they were called as Shear Walls. Papers have been written in the 1950s about analysis and design of such walls with or without openings (door/windows).

The concrete walls used in tall buildings for resisting lateral loads caused by ground motions due to earthquake were also then called as Shear Walls. However, it can be seen by simple computer analysis that in multistoreyed buildings since the height to length (span/depth) ratio of the walls is much greater, their shear deformations do not form a significant part of the total deformations. Still, the name Shear Walls stuck.

Thus, any vertical element which resists lateral loads can be a Shear Wall. It can have (in plan) rectangular, C-shape, U-shape or tubular (cylindrical) or any other shaped cross section. Depending on their cross sectional dimensions some may be more appropriately called cols. also. Of course Shear Walls can be coupled with other walls/cols. in a building to form a composite frame to resist lateral loads even more effectively. Depending on the dimensions of walls, cols., connecting beams etc. these walls can then resist overturning moments of lateral loads by a couple formed by up/down axial forces between them and other walls/cols besides flexural moment.
Vasant S. Kelkar