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THE EFFECTS OF ARCHITECTURAL-STRUCTURAL NON-CONFORMANCE OF THE REINFORCED CONCRETE FRAME BUILDINGS IN THE PRELIMINARY SEISMIC DESIGN STAGE (STATE OF THE ART)

BY

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Abstract. Reinforced Concrete (RC) Frame structures are one of the main Seismic-Resistant (SR) structural systems used by structural engineers. In these conditions, the current research study highlights the importance architectural-structural conformation of Moment-Resisting (MR) RC Frame structures in preliminary seismic design stage, so that in the current practice often results MR RC Frame buildings with increased sensitivity to earthquake. Thus, the seismic response of this structural system type required to real earthquakes was studied in parallel with seismic design specifications for RC structures, developed in P100-1 and EC 8 seismic norms. The observations regarding the effects of the inadequate architectural-structural conformation of the MR RC Frame structures overlapped with examples of real structures executed in the Iasi area (Romania). Thus, it was concluded that non-compliance with the conditions of architectural-structural conformation of MR RC Frame systems leads to seismic design of the

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building for lower values of seismic action and non-existent high ductility of the structure. In these conditions, the lateral structure develops a non-ductile seismic response with local and global collapse mechanisms (general torsion; weak floor/ ground floor).

Keywords: Earthquake, seismic norms, seismic response, general torsion, modal analysis.

1. Introduction

The present research study paper emphasizes the importance of architectural-structural conformation of Reinforced Concrete (RC) Frame structures in preliminary seismic design stage. This issue appears as a necessity to elucidate the various seismic energy dissipation mechanisms for the real structures required to a severe earthquake.

Thus, the seismic response of this structural system type required to 1971 San Fernando Earthquake (Building Science Series 40, 1971), 1978 Miyagi-ken-oki Earthquake (NBS Special Publication 592, 1980), 1994 Northridge Earthquake (NIST Special Publication 862, 1994) and 1995 Hyogoken-Nanbu (Kobe) Earthquake (NIST Special Publication 901, 1996) is presented in parallel with seismic design specifications for structures, developed in P100-1 (P100-1, 2013) and EC 8 (EC8, 2004) seismic norms.

The observations regarding the effects of the inadequate architectural-structural conformation of the MR RC Frame structures overlapped with examples of real structures executed in the Iasi area (Romania) (see Fig. 1).

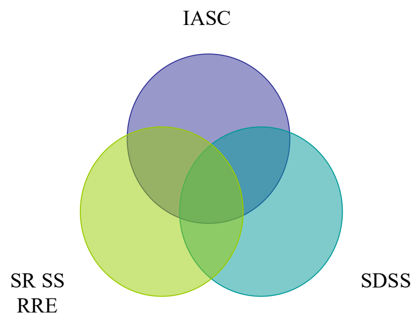


Fig. 1 – Representation of the logical scheme regarding the research topic, where: SR SS RRE is **S**eismic **R**esponse of MR RC Frame **S**tructural **S**ystem **R**equired to **R**eal **E**arthquakes (1971 San Fernando Earthquake, 1978 Miyagi-ken-oki Earthquake, 1994 Northridge Earthquake, 1995 Hyogoken-Nanbu (Kobe) Earthquake); SDSS – **S**eismic **D**esign **S**pecification for **S**tructures developed in P100-1 and EC 8 seismic norms; IASC – **I**nadequate **A**rchitectural-**S**tructural **C**onformation for MR RC Frame structures executed in Iasi area (Romania).

2. Seismic response of the RC frame systems required to real earthquakes

2.1. Los Angeles Olive View Medical Center related to 1971 San Fernando Earthquake

The 850-bed Los Angeles Olive View Medical Center, costing approximately $23.5 million, was dedicated in November, 1970. The February 9, 1971, earthquake caused three deaths, evacuation of the structure, and a total loss of the facility (Building Science Series 40, 1971).

The Medical Center was constructed generally of reinforced concrete and was designed during 1965 under the 1965 Los Angeles County Building code. Plan sheets are dated January 1966, well before the County Code was amended (November, 1966). Although the buildings were "modern", they were not constructed under the latest earthquake design requirements. The County Code as amended in 1966 was significantly different (with respect to column design and allowable concrete shear stress) from the County Code used in design. Plans and design computations for the Medical Center were provided by the Department of County Engineers, County of Los Angeles (Building Science Series 40, 1971).

A site plan of the Medical Center is shown in Fig. 2. Structures identified are: 1. Main Building (see Fig. 2 (b)); 2. Stair Towers; 3. Assembly Building; 4. Exhaust Pavilion; 5. Warehouse; 6. Ambulance Port; 7. Psychiatric Unit; 8. Canopy from Main Building to Psychiatric Unit; 9. Power Plant (Building Science Series 40, 1971).

The main building consists of four wings-A, B, C, and D (see Fig. 2). These wings are six stories in height, including the ground story. Stair towers, separated structurally from the main building, are located at the outside ends of the four wings. These wings join to form an interior courtyard. The dotted line in Fig. 2 shows the outline of the ground story. On the north and most of the west sides, the ground story is below grade. Stair towers A, B, and D are located over the roof slab of the ground story while stair tower C is supported on footings (Building Science Series 40, 1971).

In Fig. 2 and Fig. 3 the seismic response of the Los Angeles Olive View Medical Center required to 1971 San Fernando Earthquake is represented in detail.

Thus, the following conclusions and recommendations were reached (Building Science Series 40, 1971):

- An accurate appraisal must be made of the amount of ductility which can be expected in a particular type of structural design and material;

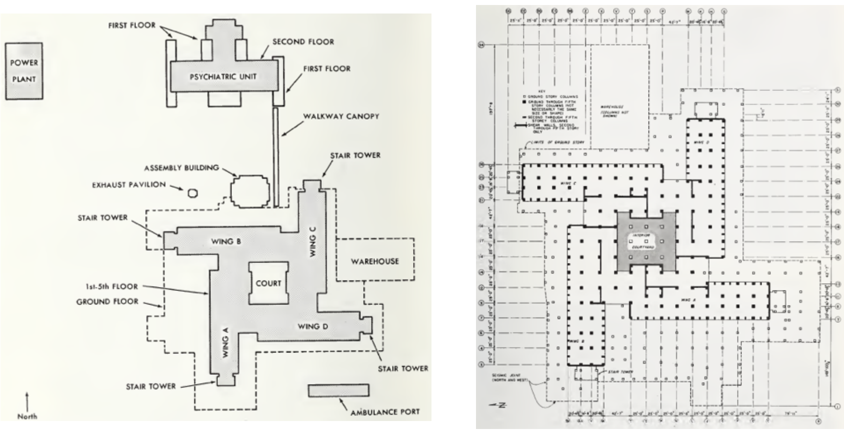
Design regulations should require material properties and connection details which are consistent with the amount of ductility allowed in dissipation of seismic energy. For example, the specific recommendations listed below to increase the ductility in reinforced concrete should be studied in detail:

a) The tied column behavior at Olive View illustrates that the maximum spacing of column ties should be limited to one-half the effective depth of the cross section to insure effectiveness as shear reinforcement;
b) Columns and piers should be designed to resist a shear at least equivalent to that imposed by a complete moment reversal on the column equal to the flexural capacity of the column. This is necessary to prevent a shear type failure since such a failure is not ductile;
c) Extra column ties or spiral wraps should be provided in the end sections of all columns and into the beam-column connections;
d) The use of lap slices for lateral reinforcement such as column ties should be discouraged. All ties should be anchored by hooks or embedment into the core in order to remain effective after covering concrete has spalled off.

• Consideration should be given to modifying the requirements for moment-resisting concrete frames to include at least some of the reinforcing requirements for ductile moment-resisting frames;

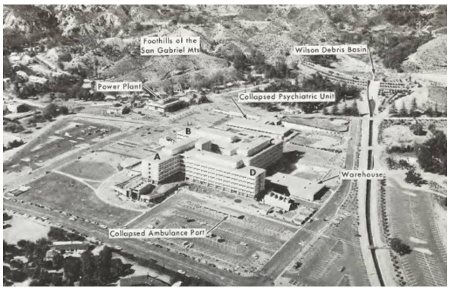
The first-story column failure of the Psychiatric Unit at Olive View illustrates the consequences of the failure of a column in shear. Once a concrete member has cracked in shear, its shear capacity becomes essentially zero. This is in contrast to the case of a flexural type of failure where the member may continue to deform but can still carry load.

Thus, to provide some shear resistance, after shear cracking of concrete, it is recommended that ties be spaced no greater than one-half the effective depth of the column cross section in moment-resisting frames as well as ductile moment- resisting frames.



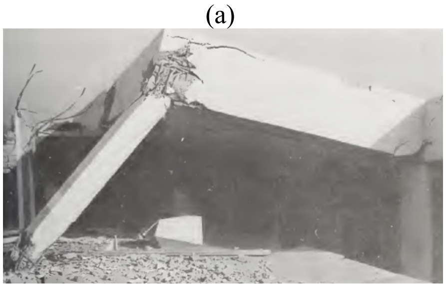
(a) (b)

Fig. 2 – (a) “Site plan for the Los Angeles Olive View Medical Center; (b) Floor plan of Main Building” (Building Science Series 40, 1971).


Foothills of the
San Gabriel Mts
Wilson Debris Basin
Power Plant
Collapsed Psychiatric Unit
B
C
A
D
Warehouse
Collapsed Ambulance Port

(a)

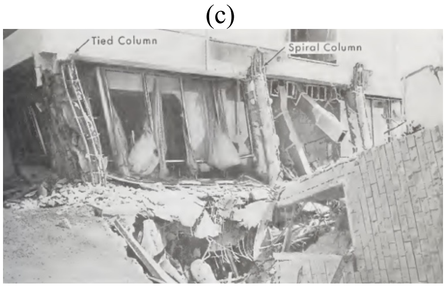
(b)



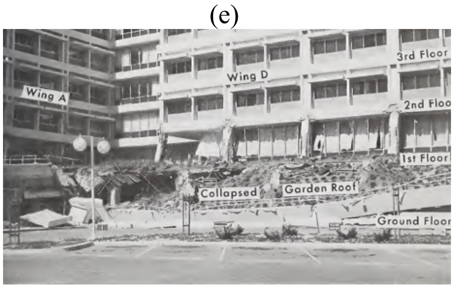
(c)


Roof
5th Floor
4th Floor
3rd Floor
2nd Floor
1st Floor
Main Entrance

(d)


Tied Column
Spiral Column

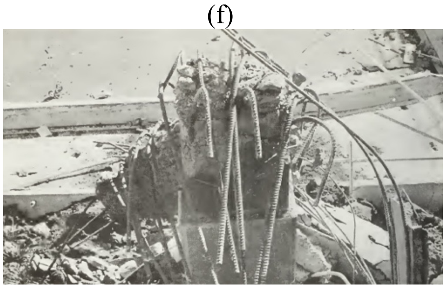
(e)


3rd Floor
Wing D
Wing A
2nd Floo
1st Floor
Collapsed
Garden Roof
Ground Floor

(f)

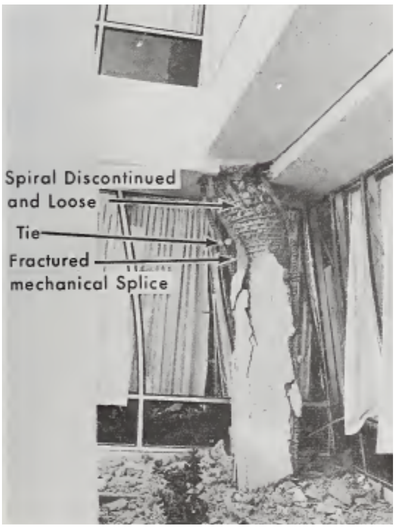

4th Floor
Wing C
3rd Floor
2nd Floor
Collapsed Garden Roof

(g)



(h)

(i)



(j)

(k)

Fig. 3 – Details regarding seismic response of the Olive View Medical Center after the 1971 San Fernando Earthquake: (a) "Olive View Medical Center after the 1971 San Fernando Earthquake; (b) South and west elevations of the Psychiatric Unit (see Fig. 2 and Fig. 3 (a)). The ground story of this building was totally collapsed. What appears to be the ground floor was the first floor of this building; (c) Damage to the canopy at the front (north) entrance to the Psychiatric Unit; (d) West face of Wing A. Note the extensive damage at the first floor and the lateral displacement of the building to the north; (e) West end of Wing B. Note the contorted column and the extent of the northward drift of the building. In the right foreground is the base of the toppled stair tower. Note that this stair tower has collapsed through the roof of the ground story; (f) South face of Wing D. Note that the bulk of major damage suffered by the building is limited to the ground, first and second floors; (g) A portion of the east face of Wing C. The collapse of the garden roof as well as the extensive damage of the first floor columns can be seen. Note the lateral displacement of the building to the north, in the first story and diagonal tension cracks in the second-story column; (h) Close-up view (looking south) of one of the failed ground-story columns which originally supported the first-floor garden roof in front of Wing D; (i) South row of first story columns, in Wind D. Note the lack of ties on the corner bars which have broken out of the columns; (j) Damaged Spiral Column; (k) Wing D after stair tower overturned" (Building Science Series 40, 1971).

2.2.(a) Maruyoshi Building related to 1978 Miyagi-ken-oki Earthquake

This was a three-story reinforced concrete frame building, one span by three, with the long axis of the building running in the E-W direction. There was a 2 m overhang on the north side of the structure (see Fig. 4 (a)). The wall shown at the west end was continuous from the second-story on up. The building had a flexible first-story (NBS Special Publication 592, 1980).

The building suffered extensive damage to the first-story (Fig. 4 (b)) and as shown in Fig. 4 (c) was tilting toward the north. The first story columns on the south side of the building failed in shear due to the strong N-S motion. A detail of the beam-column joint where the second-story floor beam frames into the first-story column is shown in Fig. 4 (d). Column ties appeared to be spaced at about 300 mm. The shear wall on the west end of the structure appeared to be undamaged. Unfortunately, this caused the structure to behave as an inverted pendulum and caused the first-story columns to accept even more shear. Note the use of the undeformed bars in the beam-column joint (Fig. 4 (d)). The building was being demolished at the time it was visited (NBS Special Publication 592, 1980).

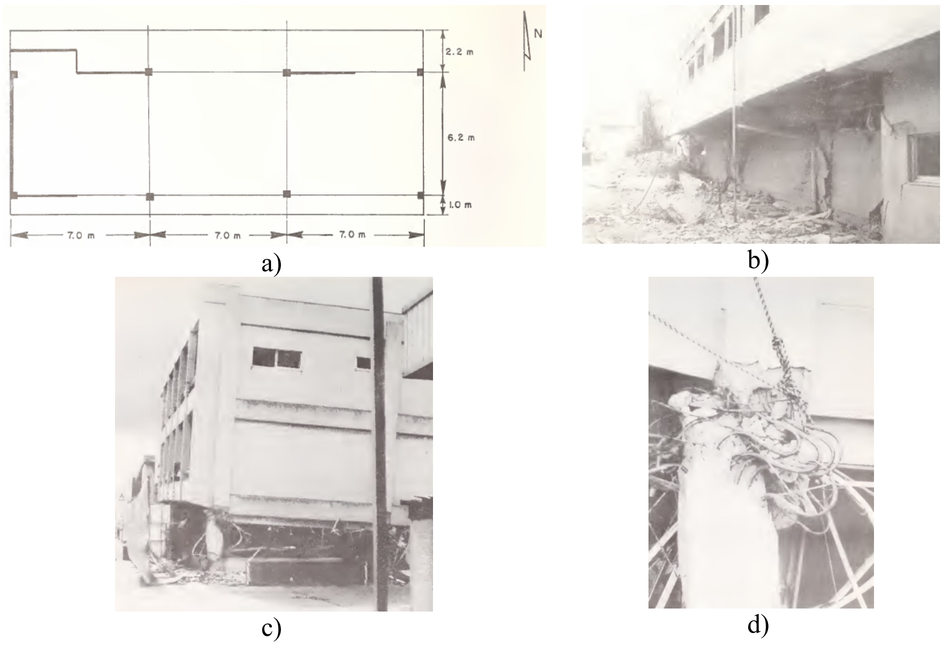
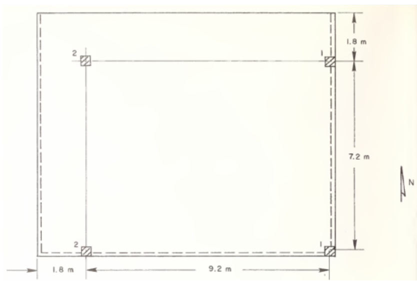


Fig. 4 – Details regarding seismic response of the Maruyoshi Building after the 1978 Miyagi-ken-oki Earthquake: "(a) Plan view of Maruyoshi Building, Oroshicho area. Note overhang to the north; (b) Maruyoshi Building, view from the northwest. This three story reinforced concrete frame, one span by three, suffered almost total collapse of the first floor; (c) Maruyoshi Building, view from the east. Failure of first story columns occurred in shear; (d) Maruyoshi Building. Detail of a joint at the top of a first story column" (NBS Special Publication 592, 1980).

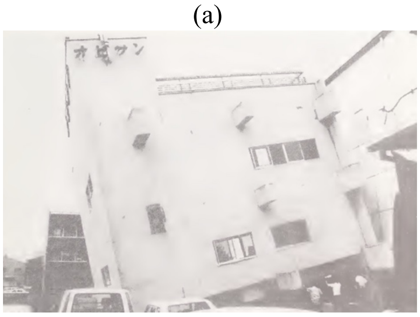
2.2.(b) Obisan Building related to 1978 Miyagi-ken-oki Earthquake

„This was a three-story reinforced concrete frame building with one span in each direction (Fig. 5 (a)). The portion of the building which overhung the column line by about 2 m contained a heavy stairwell. There were no openings in the overhanging part of the building, which added to the eccentric mass. Coupled with the strong N-S motion, this caused a failure in torsion as the columns at the west end of the building were unable to withstand the forces resulting from the large eccentric mass. Fig. 5 (b) and Fig. 5 (c) show the Obisan Building after the earthquake viewed from the north and south, respectively. It may be observed that the upper stories rotated almost as a rigid body when the first floor columns failed. The first-story of the building had been used as a display area, and was completely open. A detail of the SE column is shown in Fig. 5 (d) and in the plan. As shown in Fig. 5 (e), the first floor support beams were practically resting on the ground at the west end of the building. The walls above the first-story suffered extensive cracking during the earthquake, but remained intact (Fig. 5 (f)). The reinforcing bars that can be seen are undeformed" (NBS Special Publication 592, 1980).



(a)

(b)



(c)

(d)

(e)

(f)

Fig. 5 – Details regarding seismic response of the Obisan Building after the 1978 Miyagi-ken-oki Earthquake: "(a) Plan details of Obisan Building, Oroshicho area; (b) Obisan Building, viewed from the North. This three story reinforced concrete frame building had a heavy eccentric stairwell cantilevered over the column line at the right end of the building; (c) Obisan Building, viewed from the south. The upper stories rotated as a rigid body when the first floor column support was lost; (d) Obisan Building, detail of southeast column; (e) Obisan Building, detail of southwest column adjacent to overhang; (f) Obisan Building, south face. Walls above first story remained essentially intact" (NBS Special Publication 592, 1980).

Thus, the following conclusions and recommendations were reached (NBS Special Publication 592, 1980):

- Structure should be laid out so that they are as symmetrical as is consistent with their function. Practically all severely damaged or collapsed engineered structures exhibited a significant asymmetry of some kind in the form of an eccentric mass or placement of shear walls;
- When infill panels or spandrels are used, particular attention should be paid to detailing and to the design of structural members connected to them. When used on only one side of a building, their stiffening effect causes most of the damage to occur to that side;
- Performance of buildings clearly is related to quality control and design detailing. There were cases of failures where column ties appeared to be minimal and improperly anchored. In other instances, they appeared to be

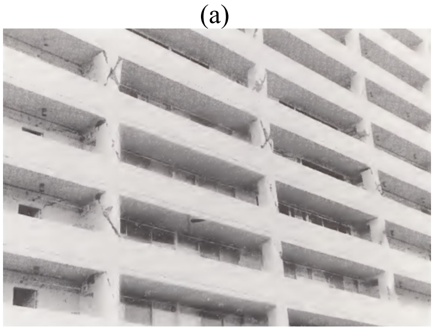
adequate. The question arises whether discrepancies exist between the structure as built and the code recommendations.

2.3. Buildings related to 1994 Northridge Earthquake

In Fig. 6 it is presented a few MR RC Frame structures required to 1994 Northridge Earthquake and their collapse/ rupture mechanisms.

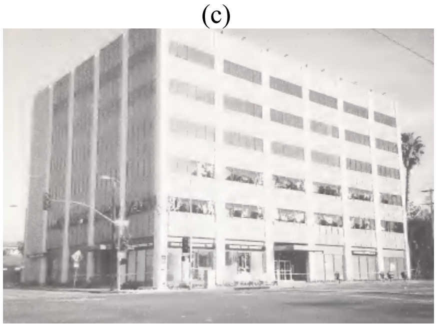
(a)

(b)



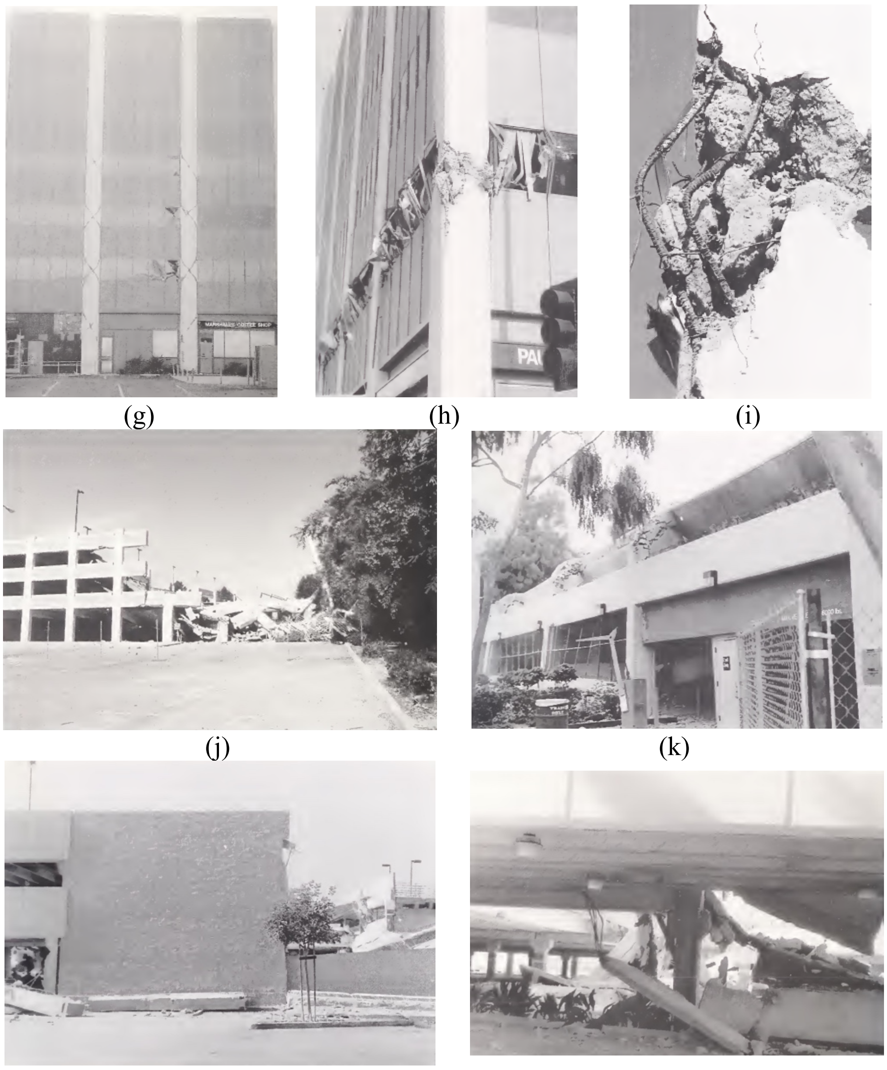
(c)

(d)



(e)

(f)



(g) (h) (i)

(j) (k)

(l) (m)

Fig. 6 – Details regarding seismic response of the few MR RC frame structures required to 1994 Northridge Earthquake: “(a) The central row of concrete first-story columns in a three story apartment building on Sherman Way in Van Nuys collapsed, causing the wood-framed upper stories to settle inward; (b) This overhead view of the partially-collapsed apartment building on Sherman Way shows the solid concrete masonry block perimeter walls along the longitudinal axis of the building. Access to the first story parking was limited to the transverse ends of the building. When the row of interior columns failed, both sides of the building settled in towards the middle; (c) The back of the Champagne Towers building was badly damaged. The railings here are solid infill, which effectively stiffen the lower half of the columns. The shortened upper half of the columns

were forced to accommodate all the displacement, and they failed in shear; (d) A closer view of the failed columns at the back of Champagne Towers shows minimal transverse reinforcement; (e) The six-story L-shaped concrete-frame Barrington Building on Olympic Boulevard in West Los Angeles lost its vertical-load carrying capacity at the second story; (f) The three-year old four-level parking garage at the Northridge campus of California State University partially collapsed in the earthquake. The collapse apparently started at the interior, causing the exterior frame to be pulled in to the middle of the structure. Precast concrete "trees" were connected to form a moment-resisting frame at the exterior of the structure; (g) The stiff infill panels of the Barrington Building effectively shortened the concrete columns, forcing all the displacement into the column sections bounded by windows. Diagonal X-cracking decreased in severity towards the top of the building; (h) The cracking in the concrete columns at the second story was so extensive that the vertical-load carrying capacity of the building was affected. The building settled 50-100 mm at the second story, as these bulging window frames attest. The worst damage was at the southeast corner, the outside corner of the L-shape, pictured here. The building, which developed a slight list, was demolished within days of the quake; (i) A close up of the damaged southeast corner of the Barrington Building shows the widely spaced, undeformed bars used for transverse reinforcement; (j) Some interior columns of the Northridge campus parking garage remained standing; (k) The cast-in-place concrete parking garage at the Kaiser Permanente Hospital complex in West Los Angeles collapsed in on itself. The moment frame that provided lateral resistance on the south face of the structure could not resist the out-of-plane forces apparently caused by the collapse of interior elements; (l) At the Northridge Fashion Center, major portions of the three-level south parking garage collapsed. The garage was precast concrete with intermittent shear walls. Most of the shear walls remained standing, largely undamaged, indicating that the lateral load was not adequately transmitted to these walls; (m) In precast parking garages, the support of the double-tees that comprise the floors is a vulnerable link in the system. This garage, at the north end of the Northridge Fashion Center, illustrates a common failure: the double-tees pulled off their supports, leaving the supporting elements essentially intact" (NIST Special Publication 862, 1994).

Thus, the following conclusions and recommendations were reached (NIST Special Publication 862, 1994):

- Soft first stories due to open parking space with few walls below stiffer upper stories are vulnerable to excessive distortion and collapse;
- Structural elements that are not considered a part of the lateral-force-resisting system can fail and initiate collapse if they are not detailed to accommodate drift;
- The unique configuration problems of parking garages (ramps, short columns, incomplete diaphragms) suggest that they need special design consideration.

Principal suggestion (recommendation):

• Develop improved requirements and design guidelines for specific building types and systems.

2.4. Buildings related to 1995 Hyogoken-Nanbu (Kobe) Earthquake

In Fig. 7 – Fig. 9 it is presented a few MR RC Frame structures required to 1995 Hyogoken-Nanbu (Kobe) Earthquake and their collapse/ rupture mechanisms.

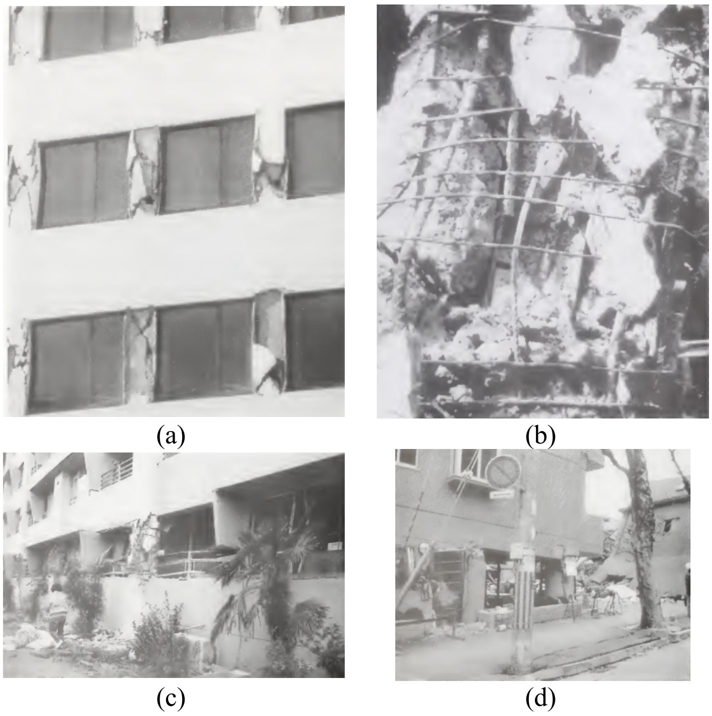
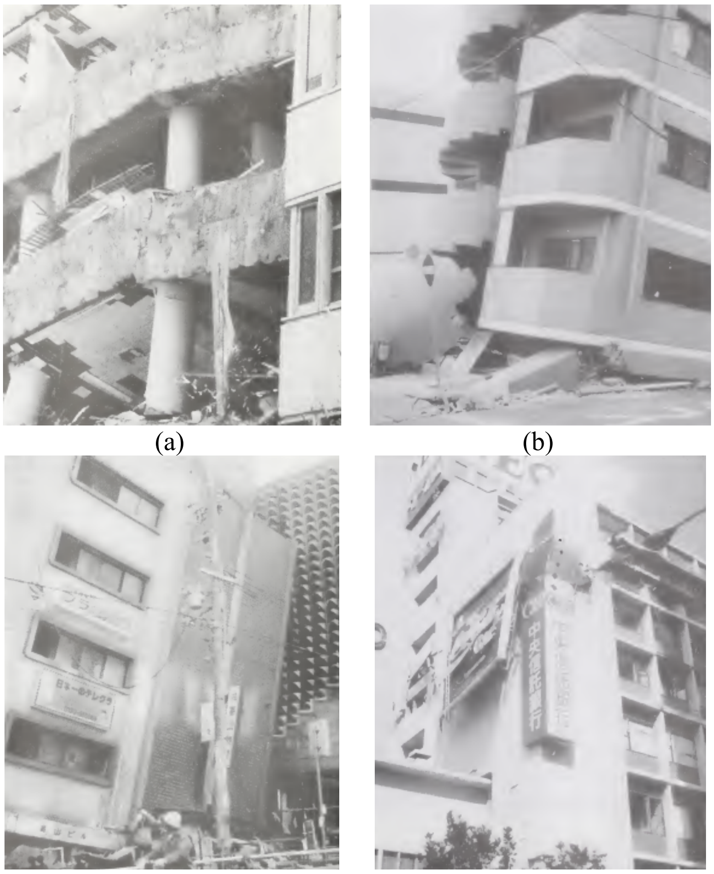


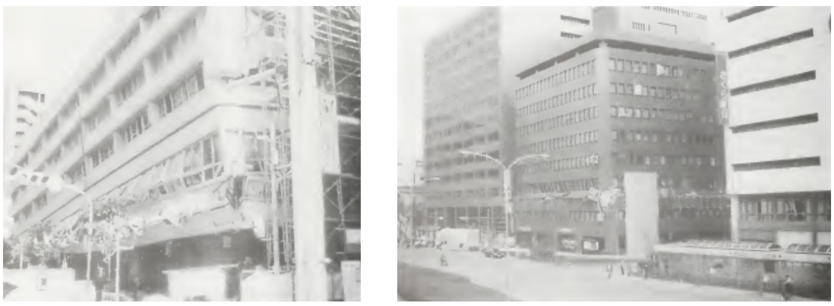
Fig. 7 – (a) “Short column effects-Otaki Building; (b) SRC column – Fukae Mitsuku Housing Project; (c) Short column effects - Fukae Mitsuku Housing Project; (d) Collapsed first story near JR Line – apartment buildings (Chuo ward)” (NIST Special Publication 901, 1996).



(a) (b)

(c) (d)

Fig. 8 – (a) “Strong beam-weak column failure; (b) Collapsed first story – commercial building in downtown Kobe; (c) Collapsed first story – Downtown Kobe; (d) Upper story collapse – Downtown Kobe” (NIST Special Publication 901, 1996).



(a) (b)

(c)

(d)

Fig. 9 – (a) "Second story collapse – commercial building in downtown Kobe; (b) Third story collapse – Downtown Kobe; (c) Upper story collapse – Downtown Kobe; (d) Totally collapsed building on Flower Road" (NIST Special Publication 901, 1996).

Thus, the following conclusions and recommendations were reached (NIST Special Publication 901, 1996):

- Damages to older reinforced concrete buildings – Many older reinforced concrete (RC) moment frame buildings experienced full or partial floor collapses;
- Good performance in new RC buildings – No evidence of collapse in any of the newer (post-1981) engineered RC structures was evident;
- Avoiding stiffness or mass irregularities – The earthquake again pointed out the need for avoiding the use of "soft" stories and stiffness or mass irregularities either in plan or elevation. Many of the older building collapses could be at least partially attributed to the presence of such weak links.

It is noted that the topic of the research study is based on graphic representations of the structures. In these conditions, a tabular synthesis of the information is not possible.

The graphic representations of the figures are of medium size and multiple in number precisely to observe as clearly as possible the effects of earthquakes and the seismic energy dissipation mechanisms recorded by buildings.

It was also considered necessary to include the information between quotation marks in order to comply with the norms of academic ethics regarding the use of existing information (for plagiarism).

3. Seismic design specifications regarding architectural-structural conformation for structures in accordance with P100-1 and EC 8 norms

The basic conceptual aspects refer to: structure simplicity; structure redundancy; structure and building geometry, in its entirety; lateral strength and stiffness, in any direction; making the slabs as horizontal diaphragms; making adequate foundations (P100-1, 2013), (EC8, 2004).

Structure simplicity presupposes the existence of a continuous and sufficiently strong structural system that ensures a clear, as direct and uninterrupted route of the seismic forces, regardless of their direction, as far as the foundation ground.

Structure redundancy involves ensuring the lateral stability of the structure in the conditions of structural element collapse and ensuring the plasticizing mechanism with sufficient plastic zones.

Structure geometry (configuration) implies the achievement of a regular structure, uniform in plan and in vertical direction.

Structure stiffness and strength to translation in two directions implies the arrangement in the plan in an orthogonal system of the structural elements that have sufficient strength and rigidity in two directions as well as the arrangement in vertical direction on the whole height of the building, of the structural elements (e.g: walls) with increased stiffness.

Torsional stiffness and strength means limiting the twisting motion of the structural system by arranging sufficiently rigid and resistant elements on the perimeter of the construction.

The realization of the slabs as horizontal diaphragms presupposes the correct conformation of the slabs so as to ensure the collection of the inertial forces and their transmission to the vertical elements of the structure as well as their solidary and coordinated engagement.

Criteria for structural regularity in the plan:

- The construction must be approximately symmetrical in plan in relation to 2 orthogonal directions, in terms of lateral stiffness distribution, strength capacities and masses;
- The construction must have a compact shape with regular contours. If the plan shape is irregular, with discontinuities that can produce significant additional stresses, it is recommended to section the construction through seismic joints, so that for each section to reach a regular shape with advantageous distributions of volumes, masses and stiffnesses;
- In multi-storey buildings, the overall reductions from one level are made on columns / walls, avoiding the support of the columns on the beams;
- The stiffness of the slabs in their plane must be high enough compared to the lateral stiffness of the vertical structural elements, so that the deformation of the slabs has a negligible effect on the distribution of horizontal forces between the vertical structural elements;
- A construction can be considered regular, with a moderate sensitivity to global torsion, if the maximum displacement recorded on the perimeter of the construction in the seismic design combination, in the direction

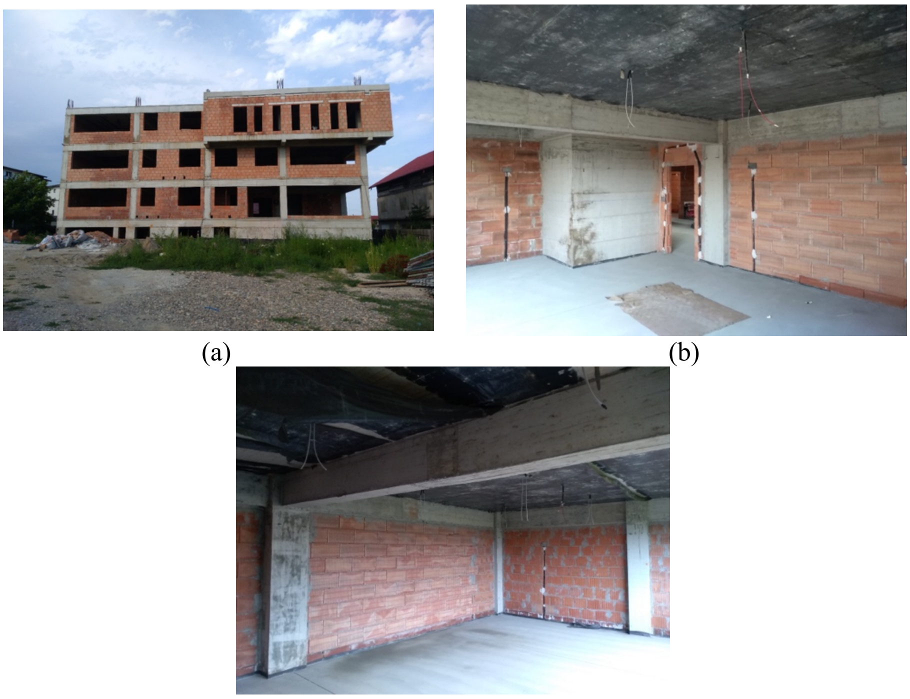
of force, does not exceed by more than 35% the average of maximum and minimum displacements.

Criteria for vertical regularity:

- The structural system develops monotonously vertically, without significant variations from the foundation to the top of the building. Gradual withdrawals are allowed on the height of the building if they do not exceed, at any level, 20% of the size from the level immediately below;
- The structure does not show, at any level, reductions in lateral stiffness greater than 30% of the stiffness of the level immediately above or immediately below (the structure has no weak storeys);
- The structure does not have, at any level, a lateral strength by more than 20% lower than that of the level immediately above or below (the structure does not have weak storeys in terms of lateral strength);
- If the dimensions of the structural elements are reduced from the base to the top of the structure, the variation in stiffness and lateral strength must be uniform, without sudden reductions from a lower level to a higher level;
- The masses applied to the construction are evenly distributed. This means that at no level is the corresponding mass more than 50% higher than the masses applied at adjacent levels. The exception to this rule is the situation in which the additional masses are concentrated at the base of the structure;
- The structure has no vertical discontinuities that divert the path of forces to the foundations. The provision refers both to deviations in the same plane of the structure and to deviations from one plane to another vertical plane of the construction.

4. Real MR RC frame structures executed in Iasi area (Romania) with inadequate architectural-structural conformation

The structural system represented in Fig. 10 has balconies in one corner of the building, a rigid attachment of the elevator shaft walls to the reinforced concrete frame beams as well as rigidly infilled masonry walls in the RC frames.

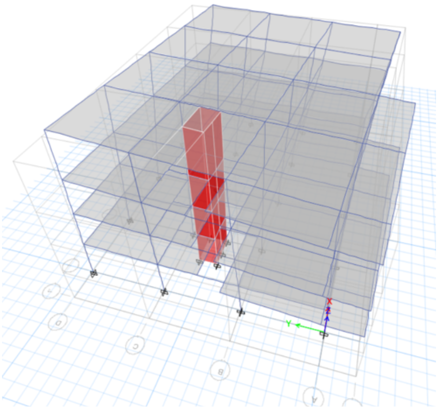


(a) (b)

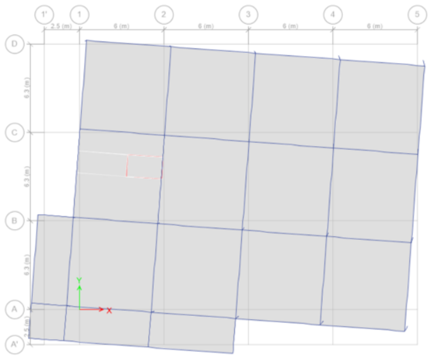
(c)

Fig. 10 – Representation of MR RC frame structure: (a) with the mass concentration of the structure at the top storey on a limited surface (Divya & Murali, 2022); (b) with rigid attachment of the elevator shaft walls to the RC frame beams; (c) with rigidly infilled masonry walls in the RC frames (Tena-Colunga A., 2021).

In these conditions, the structural system develops the first torsional vibration mode (see Fig. 11) (Ghayoumian & Emami, 2020). Practically, both the vertical regularity criteria and the structural regularity criteria in the plan (specified in the P100-1 and EC 8 seismic norms) are not respected (Khanal & Chaulagain, 2020). Also, see Obisan Building (Fig. 5) related to 1978 Miyagi-ken-oki Earthquake.



(a)



(b)

Fig. 11 – Representation of the first torsional vibration mode: (a) 3D view; (b) top-storey view.

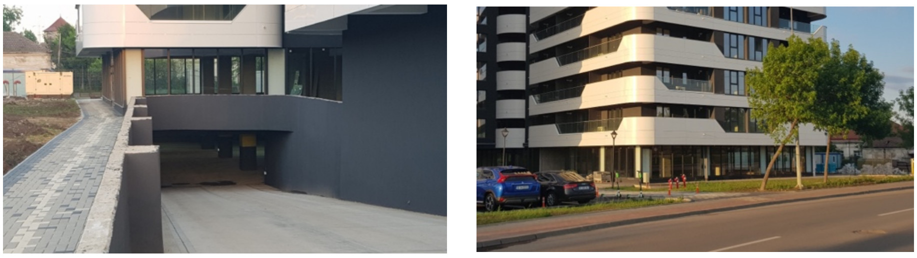
In Fig. 12 (a, b) an „H” shaped MR RC frame system is observed. The seismic response of this type of structure is represented in Fig. 3 (a), Fig. 3 (d) – Fig. 3 (i), Fig. 3 (j, k) corresponding to Los Angeles Olive View Medical Center related to 1971 San Fernando Earthquake. Also, these two structural RC frame systems have a basement on the perimeter of the building (see Fig. 12 (c)), which can lead to the rupture of the basement/ ground floor columns and the collapse of the superstructure (see Fig. 3 (a), Fig. 3 (e, f)).

In Fig. 12 (d) and Fig. 13 (c, d) shows the presence of weak first storey for MR RC frame structures for which the seismic response coincides with that for the structures represented in Fig. 7 – Fig. 9 related to Hyogoken-Nanbu (Kobe) Earthquake.

In Fig. 13 (a, b) is a MR RC Frame system with horizontal irregularity that can dissipate seismic energy through a different mechanism than that specified in the current norms for seismic design of structures P100-1 (P100-1, 2013) and EC 8 (EC8, 2004).

(a)

(b)



(c) (d)

Fig. 12 – Representation of an „H"- shaped building: (a) front view; (b) back view; (c) underground parking view; (d) ground floor view – commercial space.

(a) (b) (c)

Fig. 13 – Representation of the: (a), (b) Structural system with horizontal and vertical irregularity (Archana & Abdul Akbar, 2021), (Wang *et al.*, 2018); (c), (d) Structural system with vertical stiffness difference - weak first storey and strong box created by the rest of the upper floors (Gerasimidis *et al.*, 2012).

It is specified that it is not possible to provide results in terms of lateral displacements or lateral forces through numerical simulations because there is no access to the execution data of the buildings. Practically, a visual inspection was carried out through which the specific conclusions were established.

The solutions for the initially designed structures are not proposed. The objectives of this research study are different.

5. Conclusions

Seismic response of the RC Frame structures required to 1971 San Fernando Earthquake, 1978 Miyagi-ken- oki Earthquake, 1994 Northridge Earthquake and 1995 Hyogoken-Nanbu (Kobe) Earthquake present real ways of seismic energy dissipation mechanisms that in many situations are unfavorable to the safety of human life.

Seismic response of the RC Frame structures required to 1971 San Fernando Earthquake, 1978 Miyagi-ken- oki Earthquake, 1994 Northridge Earthquake and 1995 Hyogoken-Nanbu (Kobe) Earthquake present the effects of the inadequate architectural-structural conformation of the RC Frame structures.

The seismic design specifications regarding architectural-structural conformation of structures, developed in P100-1 and EC 8 norms clearly highlight the need to respect the basic conceptual aspects refer to: structure simplicity; structure redundancy; structure and building geometry, in its entirety; lateral strength and stiffness, in any direction; making the slabs as horizontal diaphragms; making adequate foundations.

The presented MR RC Frame structures designed in Iasi area (Romania) do not respect the architectural-structural conformation of P100-1 and EC 8 norms. Thus, it is necessary to perform a modal calculation by the structural engineer as a mandatory verification of the architectural-structural conformation of the structure, taking into account the possible non-structural elements that may influence the global / local seismic response of MR RC Frame structure.

The presented MR RC Frame structures designed in Iasi area (Romania) can dissipate seismic energy through weak ground floor mechanisms, global torsion, rupture of columns at underground floors and collapse of entire superstructure etc.

MR RC Frame structures with global torsional sensitivity must be designed for a lower „q" value than high ductility structures.

Not considering the effects of seismic response from overall torsion or weak ground floor in the stage of architectural-structural conformation of the MR RC Frame structure, there is a risk that the structure will be designed for lower values of seismic action and non-existent ductility of the structure.

The real seismic response of the constructions presented in the current research paper (Iasi area, Romania) will be recorded with certainty only during the real seismic loads at the site.

REFERENCES

*** *Cod de proiectare seismică", Partea I, Prevederi de proiectare pentru clădiri, Indicativ P100-1*, Ministerul Dezvoltării Regionale și Administrației Publice, Mai 2013.

*** *Eurocod 8: Proiectarea structurilor pentru rezistența la cutremur, Partea 1: Reguli generale, acțiuni seismice și reguli pentru clădiri, Indicativ SR EN 1998-1*, ASRO, București, 2004.

*** *Building Science Series 40, Engineering Aspects of the 1971 San Fernando Earthquake,* U.S. Department of Commerce, December 1971.

*** *NBS Special Publication 592, An Investigation of the Miyagi-ken-oki, Japan, Earthquake of June 12, 1978,* U.S. Department of Commerce, October 1980.
*** *NIST Special Publication 862, 1994 Northridge Earthquake, Performance of Structures, Lifelines and Fire Protection Systems,* U.S. Department of Commerce, May 1994.
*** *NIST Special Publication 901, The January 17, 1995 Hyogoken-Nanbu (Kobe) Earthquake,* U.S. Department of Commerce, July 1996.
Archana A.R., Abdul Akbar M., *Structural irregularity quantification in buildings using vital signs*, Structures **34** (2021) 2592-2599, doi: https://doi.org/10.1016/j.istruc.2021.09.026.
Divya R., Murali K., *Comparative analysis of behaviour of horizontal and vertical irregular buildings with and without using shear walls by ETABS software*, Materials Today: Proceedings **52** (2022) 1821-1830, doi: https://doi.org/10.1016/j.matpr.2021.11.489.
Gerasimidis S., Bisbos C.D., Baniotopoulos C.C., *Vertical geometric irregularity assessment of steel frames on robustness and disproportionate collapse*, Journal of Constructional Steel Research **74** (2012) 76-89, doi: https://doi.org/10.1016/j.jcsr.2012.02.011.
Ghayoumian G., Emami A.R., *A multi-direction pushover procedure for seismic response assessment of low-to-medium-rise modern reinforced concrete buildings with special dual system having torsional irregularity*, Structures **28** (2020) 1077-1107, doi: https://doi.org/10.1016/j.istruc.2020.09.031.
Khanal B., Chaulagain H., *Seismic elastic performance of L-shaped building frames through plan irregularities*, Structures **27** (2020) 22-36, doi: https://doi.org/10.1016/j.istruc.2020.05.017.
Tena-Colunga A., *Conditions of structural irregularity. Relationships with observed earthquake damage in Mexico City in 2017*, Soil Dynamics and Earthquake Engineering **143** (2021) 106630, doi: https://doi.org/10.1016/j.soildyn.2021.106630.
Wang J., Dai K., Yin Y., Tesfamariam S., *Seismic performance-based design and risk abalysis of thermal power plant building with consideration of vertical and mass irregularities*, Engineering Structures **164** (2018) 141-154, doi: https://doi.org/10.1016/j.engstruct.2018.03.001.

IMPORTANȚA CONFORMĂRII ARHITECTURAL-STRUCTURALE A CLĂDIRILOR TIP CADRU DE BETON ARMAT ÎN ETAPA PRELIMINARĂ DE PROIECTARE SEISMICĂ

Structurile tip cadru de beton armat reprezintă una dintre principalele sisteme structurale seismo-rezistente utilizate de inginerii structuriști. În aceste condiții, prezentul studiu de cercetare scoate în evidență importanța conformării structurii de arhitect și inginer în etapa preliminară de proiectare, încât în practica curentă deseori rezultă clădiri cu sensibilitate sporită la seism. Astfel, s-a studiat paralel atât răspunsul seismic al acestui tip de sistem structural solicitat la cutremure reale, cât și specificațiile normativelor de proiectare seismică a structurilor P100-1 și EC 8. Observațiile privind efectele conformării arhitectural-structurale necorespunzătoare ale structurilor seismo-rezistente tip cadru de beton armat s-au suprapus cu exemple de structuri reale executate în zona Iași (Romania). Astfel, s-a concluzionat că nerespectarea condițiilor de conformare arhitectural-structurală a sistemelor seismo-rezistente tip cadru de beton armat conduce la proiectare seismică a clădirii pentru valori inferioare a acțiunii seismice și pentru înaltă ductilitate inexistentă a construcției. În aceste condiții, structura laterală dezvoltă un răspuns seismic neductil cu mecanisme de colaps local și global (torsiune de ansamblu; parter/ etaj slab).