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CHALLENGES OF STRUCTURAL DESIGN IN SEISMIC REGIONS: EXAMPLES FROM TWO CASE STUDIES

Summary: This paper presents two building projects in seismic regions: Project A (Italy) and Project B (Middle East). It examines challenges in aligning architectural intent, client expectations, regulations, and seismic performance throughout design. Project A, designed to Italian NTC18, adopted an elastic design approach due to architectural constraints limiting ductility, requiring strict control of force demand and material behavior. Project B, a high-importance, geometrically irregular nine-building complex, required post-earthquake operability. Seismic isolation proposed at concept stage was rejected, leading to a conventional solution with limited ductility and a predominantly elastic composite system (steel encased in concrete). The paper highlights how constraints, risk perception, and interdisciplinary coordination shape seismic design strategies.

Keywords: seismic design, architectural constraints, elastic design

IZAZOVI PROJEKTOVANJA KONSTRUKCIJA U SEIZMIČKIM PODRUČJIMA: DVA PRIMERA IZ PRAKSE

Rezime: Rad prikazuje dva projekta zgrada u seizmičkim područjima: Projekat A (Italija) i Projekat B (Bliski istok). Analiziraju se izazovi usklađivanja arhitektonskog koncepta, zahteva investitora, propisa i seizmičkih performansi tokom projektovanja. Projekat A, projektovan prema standardu NTC18, zasnivao se na elastičnom pristupu zbog arhitektonskih ograničenja koja su smanjivala mogućnost obezbeđenja duktilnosti, pa je bila neophodna stroga kontrola seizmičkih sila i ponašanja materijala. Projekat B, kompleks od devet geometrijski nepravilnih objekata velike važnosti, zahtevao je funkcionalnost nakon zemljotresa. Nakon odbacivanja seizmičke izolacije usvojeno je konvencionalno rešenje sa ograničenom duktilnošću i pretežno elastičnim spregnutim sistemom. Rad pokazuje kako ograničenja i koordinacija učesnika utiču na strategije seizmičkog projektovanja.

Ključne reči: seizmičko projektovanje, arhitektonska ograničenja, elastični dizajn

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1. INTRODUCTION

Earthquake-resistant behaviour is governed less by the strength of any single member than by the overall layout and load path of the structure. Regularity in plan and elevation, lightness, a clear and symmetric distribution of mass and stiffness, short and direct load paths, and the capacity to deform in a ductile manner, without sudden loss of strength, are the qualities that allow a building to absorb seismic energy reliably. Where these are present the response is predictable and the demands on individual elements are modest; where they are absent, force concentrations, torsional response, and soft-storey mechanisms arise, and the structure must compensate through size, detailing, or restraint.

Seismic codes are written, in large measure, for the idealised structure: regular, repetitive, and well-behaved, with the simplifying assumptions of the design methods broadly satisfied. Real buildings, and particularly those shaped by ambitious architectural intent, rarely conform to this ideal. Transfer structures, setbacks, large openings, slender forms, and discontinuous load paths place a building outside the comfortable territory the codes envisage, leaving the engineer to reconcile a non-compliant geometry with prescriptive provisions through additional analysis, conservative assumptions, and judgement. This discord between what the codes assume and what is actually built is a recurring source of difficulty, and it is most acute precisely where the architecture is most expressive.

Performance objectives compound the issue. The baseline intent of most codes is life safety: the structure may be damaged, even severely, provided it does not collapse and occupants can escape. Operational continuity is a far more demanding target, requiring that the building, its structure, and often its services remain usable immediately after the design earthquake. Achieving it generally means keeping the response close to elastic, limiting drift and damage, and protecting non-structural systems, all of which carry significant implications for stiffness, member size, and cost that are not always apparent when the objective is first set.

Those implications are frequently underestimated by clients, whose risk perception drives an expectation of a fully operational facility after a major event without full appreciation of what that entails. The consequences include inflated budgets, unconventional solutions such as base isolation or supplemental damping that may be unfamiliar in the region, or stocky, heavily reinforced designs that sit uneasily with the architectural vision. Where the brief is more ambitious than the budget or the geometry can sustain, the mismatch tends to surface late, often after tender, prompting redesign once the true cost and constructional consequences of an overzealous performance target are understood.

The two projects examined in this paper illustrate how these pressures of constraint, risk perception, and interdisciplinary coordination shape the seismic strategy that is ultimately adopted.

2. PROJECT A – ITALY (ELASTIC DESIGN UNDER CONSTRAINTS)

2.1. PROJECT BRIEF

Project A is a new residential complex in Rome, comprising eight buildings of three typological families (Types A, B, and C, see Figure 1) sharing a single underground parking level. The work covered under this project is limited to the four identical Type C buildings, which feature an irregular L-shaped plan with unequal wings and rounded corners with 6 floors above ground level (see Figure 2). The lateral force-resisting system consists of RC cores, shear walls, and columns supporting solid flat slabs, with vertical elements positioned to follow architectural constraints. Project A was led by Oneworks as lead designer, with the authors acting as structural subconsultants.

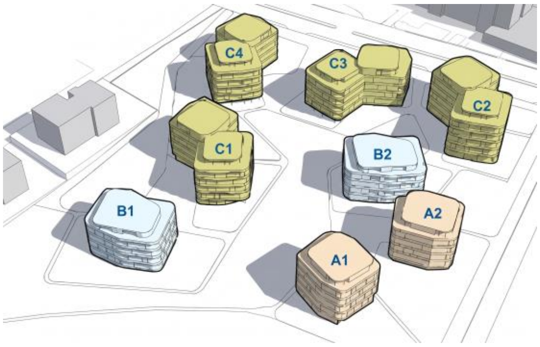

Figure 1: Building types (image courtesy of Oneworks)

The site has low-to-moderate seismicity (ag = 0.147g, 475-year return period), subsoil Category B and topographic category T1. Foundations consist of RC piled rafts, carefully coordinated to avoid existing piles from the demolished building previously occupying the site.

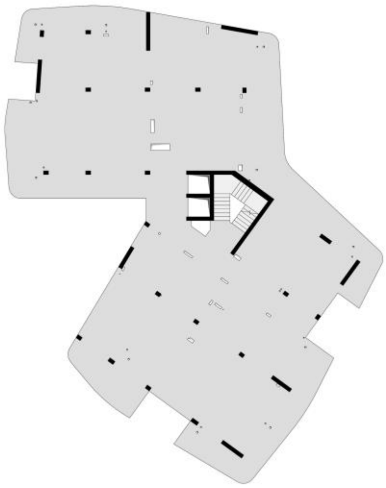

Figure 2: Typical floor plan (image courtesy of Oneworks)

2.1.1. Applicable codes and regulatory framework

The project was designed in line with the Italian Technical Standards for Construction (NTC2018) [1] and European (Eurocodes) codes and regulations. The principal difference, from seismic design perspective, between NTC 2018 and EN 1998 is that NTC 2018 allows the adoption of elastic strength-based design approach for moderate seismicity [1], which EN 1998-1 explicitly discourages [2].

2.1.2. Architectural constraints limiting ductility

Due to strict architectural requirements, the walls could not be arranged symmetrically, resulting in significant torsional eccentricity between the centre of mass and the centre of stiffness. Such arrangement resulted in, from seismic perspective, undesirable torsional modal behaviour.

2.2. DESIGN APPROACH

The building is classified as torsionally flexible, limiting the dissipative behaviour factor to q = 2.0 (Ductility Class B, equivalent to DCM in EN 1998-1).

NTC2018 differs from EN 1998 in seismic elastic design philosophy. While EN 1998 states that elastic design is recommended only for seismic regions with low seismicity, NTC 2018 has no such restrictions. On the other hand, NTC 2018 has defined stricter requirements to satisfy.

If dissipative design is pursued, under NTC2018 §7.4.4.5.1, the concrete web compression resistance in dissipative zones would be reduced by a factor of 0.4, while design shear forces are simultaneously amplified per Eq. 7.4.14. For this building, the combination of these two effects with the high torsional shear in the cores would require wall thicknesses that are architecturally unacceptable, as presented in the following chapters, thus the design approach was selected to be non-dissipative.

2.2.1. Code provisions

For this torsionally flexible structure, the non-dissipative behaviour factor per NTC 2018 §7.3.1 [1] is q_{ND} = 1.33.

For non-dissipative structures, design under seismic load combinations requires that concrete and reinforcement strains remain within the elastic range — i.e. below ε_{c0} and ε_0, respectively — as illustrated in Figure 3 and required by the authoritative commentary [4].

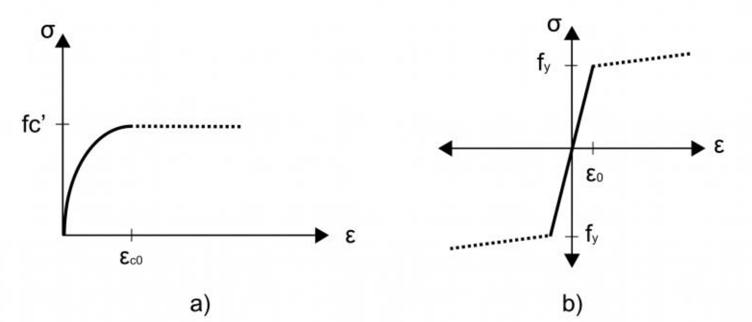

Figure 3: Stress-strain diagrams for seismic design: a) concrete, b) reinforcing steel

Beam-column node checks per NTC2018 §7.4.4.3 remain mandatory for non-dissipative structures, applying the same formulae as for Ductility Class B: node shear failure is a brittle mechanism that would invalidate the assumed structural scheme whether the design is dissipative or not.

2.3. IMPLICATIONS OF SELECTED DESIGN APPROACH

2.3.1. Force levels comparison

The adoption of q = 1.33 reduces the design response spectrum to approximately 75% of the elastic spectrum, translating into seismic design forces roughly 50% higher compared to a conventional dissipative design (q = 2.0). The seismic load combination philosophy remains unchanged. The design implications are illustrated on representative wall sections at ground level in Tables 1 and 2.

Table 1 analyses shear capacity and force levels of a core wall. The wall section is 300mm thick, 5300mm long with horizontal rebar of Φ16-100. Concrete is C30/37 and rebar characteristic yield strength is fyk= 450 MPa:

Table 1: Comparison between elastic and elastoplastic wall shear demand and capacity

Seismic design approach	q [-]	V_{Ed} [kN][1]	V_{Ed}' [kN][2]	V_{Rd} [kN]
Dissipative Ductility Class B	2.00	3538	6580	2190[3]
Non dissipative elastic	1.33	5321	5321	5474
Note 1: Analysis is carried out with elements of the same size, dissipative approach would require thicker elements leading to stiffer structure and higher seismic forces Note 2: Shear force increases due to formation of a plastic hinge as per NTC 2018 §7.4.14 Note 3: Shear reduction in dissipative zone as per NTC 2018 §7.4.4.5.1				

Table 1 shows that, despite the higher seismic demand of the non-dissipative approach, the amplified dissipative shear force V_{Ed}' = 6580 kN significantly exceeds the non-dissipative demand of 5321 kN. Combined with the 0.4 reduction factor on concrete compression resistance in dissipative zones, this would require up to three times thicker walls to satisfy shear verification — the primary driver for adopting the non-dissipative strategy in this project.

Table 2 compares wall major bending capacities between elastoplastic (dissipative) and elastic limited strain (non-dissipative) material behaviour. The wall section is 300mm thick, 2300mm long with vertical rebar of Φ20-150 on both faces. Concrete is C30/37 and rebar characteristic yield strength is fyk= 450 MPa:

Table 2: Comparison between elastic and elastoplastic wall major bending capacity

Seismic design approach	N_{Ed} [kN][1]	M_{Rd} [kNm]
Dissipative Ductility Class B	1722	7046
Non dissipative elastic	1722	6500
Note 1: Positive is compression		

Table 2 illustrates that, for the same axial force, the limited-strain section model of the non-dissipative approach yields a lower bending capacity than the full elastoplastic model (6500 vs 7046 kNm), as a direct consequence of the restricted stress-strain diagram.

2.3.2. Reinforcement detailing

The non-dissipative strategy eliminates most seismic detailing requirements: no curvature ductility checks, no confinement stirrups at wall boundary zones, and no capacity design hierarchy. Horizontal wall reinforcement reverts to the standard minimum of $\rho_h \geq 0.10\%$, half the Ductility Class B requirement of 0.20%. Similarly, columns require no closely spaced confinement hoops in critical regions; however, closely spaced transverse reinforcement was maintained as good practice. The exception is beam-column nodes, which must be detailed to Ductility Class B requirements regardless of the global design strategy. All elements are otherwise detailed uniformly under standard reinforced concrete rules, equivalent to those of Eurocode 2, simplifying site execution and quality assurance.

2.4. DESIGN TOOLS

Analysis and design were carried out using MidasGen, which allows the user to define custom stress-strain material curves and apply NTC2018 provisions directly in the design process. SAP2000 with the CSI ViS Concrete Design plug-in — developed by CSI Italia for post-processing of RC elements to Eurocode and NTC2018 requirements— was used as an independent verification tool [3].

2.5. LIMITATIONS AND ADVANTAGES OF NON-DISSIPATIVE APPROACH

Both dissipative and non-dissipative design approaches are valid and legitimate. The non-dissipative approach eliminates capacity design requirements, replacing complex ductility detailing with uniform reinforcement rules across all elements. The trade-off — higher seismic design forces (q = 1.33 vs q = 2.0) — is manageable in low-to-moderate seismicity, where the force increment over dissipative design is modest and does not result in impractical member sizes. For irregular structures with low behaviour factors, this is the logical choice where the dissipative behaviour factor is already reduced to q = 2.0, limiting the force reduction benefit of dissipative design. The essentially elastic response also limits inter-storey drifts and residual deformations, reducing non-structural damage and supporting post-earthquake functionality — a consideration of particular relevance for strategic buildings such as hospitals or emergency facilities.

The principal limitation concerns the hierarchy of failure modes. The limited-strain section model yields a lower bending capacity than the full elastoplastic model (Table 2), and without an enforced weak-bending / strong-shear hierarchy, shear failure may precede flexural yielding should the actual seismic demand exceed design values. Since shear is a brittle failure mode, this represents a residual risk that the designer must acknowledge.

3. PROJECT B – NINE BUILDING COMPLEX IN THE MIDDLE EAST (HIGH IMPORTANCE, IRREGULAR GEOMETRY)

3.1. PROJECT BRIEF

Project B comprises nine buildings on a radial masterplan. There are five different building typologies: one P, four W, two H, one C and one building type S. Buildings P, W and H have a high importance classification and a 100-year design life.

The client's requirement for the P building was post maximum credible earthquake (MCE) operability: it had to remain functional after the seismic event (Immediate Occupancy), effectively raising the seismic design demand. Project lies in a seismically active region, which combined with client's requirement, necessitated the building to be designed for earthquake with 1/2475-year return period.

Key constraints were unrealistic client's performance expectation for post-earthquake operability, use of stiff diagrid (see Figure 4) taking significant lateral load while having to remain elastic after earthquake and architecturally preferable slender members. Another important constraint was the client's preference of reinforced concrete over structural steel elements. The design was carried out in accordance with European (Eurocode) standards.

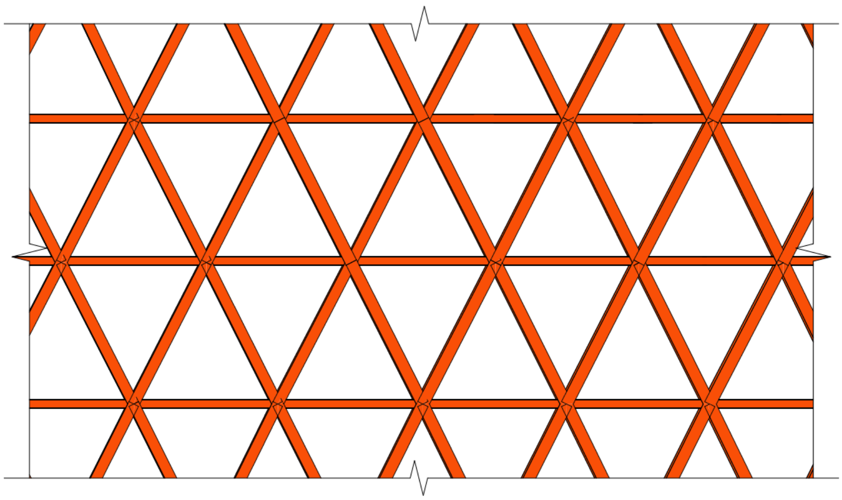

Figure 4: Diagrid system

3.2. GEOMETRIC AND STRUCTURAL IRREGULARITIES

The architectural concept produced pronounced irregularities that conflict with the regularity principles of Eurocode 8. In plan, the triangle-shaped buildings are concave and non-compact with large diaphragm openings at the central atrium (W and H) and dome (P); in elevation, cantilevered overhangs of up to 25m length, massive steel canopies arching more than the entire footprint of the building and sitting above its arched inclined concrete roof, stepped terraces, interrupted vertical elements and non-uniform mass and stiffness are present. Perimeter walls are envisaged as architecturally expressed stiff diagrid that needs to remain essentially elastic post-earthquake so that it could keep carrying gravity loads. Because of all the above, the structural system cannot be categorized as any of the code prescribed high ductility energy dissipating systems.

3.3. CONCEPT DESIGN PHASE

Base isolation was proposed as a response to the combination of high seismicity, high importance and severe irregularity. By lengthening the fundamental period and concentrating displacement and energy dissipation within the isolation interface, it can substantially reduce the superstructure forces. Dissipation occurs within the isolators rather than through damaging inelastic deformation, supporting Immediate Occupancy while preserving the architectural form and permitting slender members. Figure 5 presents storey shear force levels in one direction for base isolated and non-isolated W building model. It can be observed that loads drop by more than 60% when building is isolated. This coupled with the fact that isolated building is fully operational after earthquake makes a rather compelling case for pursuing this technical solution.

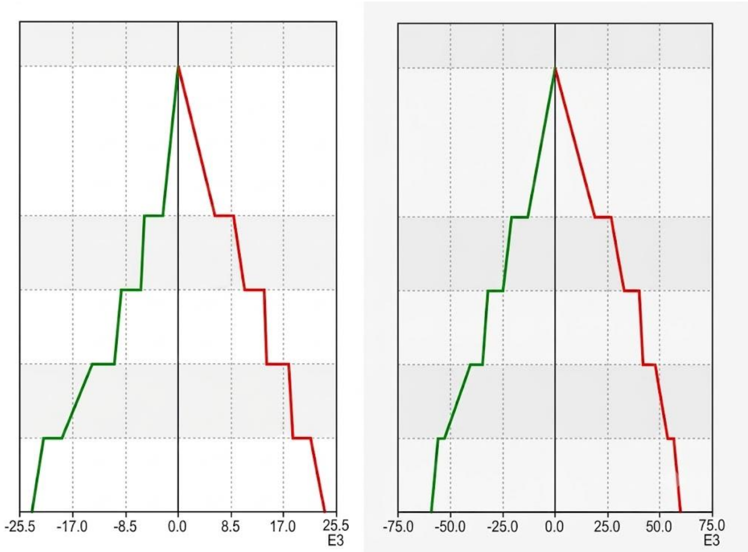

Figure 5: Storey Shear Force Levels [kN] in a) base isolated (left) and b) non-isolated (right) W building model

Note that design spectrum for base isolated structures is equal to elastic spectrum (i.e. behaviour factor q is equal to 1), while non-base isolated design spectrum utilizes a behaviour factor estimated to be equal to 2.4 (shear walls and composite diagrid).

The proposal raised concerns over constructability (specialist installation, with few experienced regional contractors), perception (unfamiliarity, maintenance and potential isolator replacement) as well as visual and functional impact (large movement joints).

3.4. SCHEMATIC DESIGN PHASE

Weighed against the constraints identified during the concept design phase, the seismic isolation was not carried forward, and the design reverted to a conventional solution.

As fully reinforced concrete solution required architecturally unacceptable member sizes, the adopted compromise lateral-load-resisting-system is a predominantly steel-concrete composite arrangement – steel sections diagrid and composite shear walls with embedded steel sections to carry the high forces while limiting member sizes. It is important to note that neither the European nor North American seismic design codes classify steel diagrid as a distinct lateral restraining system element, but existing studies indicate that it does possess some dissipative capability [5]. The key consideration is that the diagrid, as an element that supports gravity loads in addition to earthquake-induced loads, must remain elastic post the design earthquake.

Proof of dissipative assumptions requires a non-linear pushover or time history analysis which was intended for the next design stage. This approach is not uncommon, as these types of analyses are rarely performed during the coordination process, due to time restrictions and dynamic changes.

3.5. DESIGN IMPLICATIONS AND TRADE-OFFS

The adopted strategy transfers the high seismic forces into the structure, driving up member sizes – thicker cores and shear walls, heavier composite diagrid sections than the base-isolated scheme would require. Illustrating the central trade-off: code compliance and operability are achieved at the expense of efficiency and architectural freedom that base isolation could have achieved.

4. INTERDISCIPLINARY COORDINATION AND DECISION MAKING

4.1. INTERACTION BETWEEN ARCHITECT, ENGINEER AND CLIENT

In both projects, the final structural solution was governed as much by project priorities as by seismic demand. The architect established the building form and spatial constraints, the client defined the required performance level and commercial objectives, while the structural engineer was responsible for reconciling these requirements within the governing design code.

In Project A, the architectural layout fixed the location of the cores and shear walls, resulting in unavoidable torsional irregularity. Rather than attempting to alter the geometry, the engineering effort focused on selecting a seismic design strategy compatible with the imposed configuration. In Project B, the client's requirement for post-earthquake operability initially favoured seismic isolation, but concerns regarding cost, constructability and unfamiliarity with the technology ultimately led to adoption of a conventional composite structural system. These examples demonstrate that structural systems are often determined by project constraints rather than by structural optimization alone.

4.2. MANAGING CONSTRAINTS ACROSS DESIGN STAGES

Design constraints evolve throughout a project, progressively reducing the available design freedom. Decisions that remain flexible during concept design frequently become fixed during schematic and detailed design, making subsequent changes increasingly costly.

The two projects illustrate this process differently. In Project A, the architectural arrangement was accepted as an immutable constraint from the outset, leading directly to adoption of a non-dissipative design philosophy. In Project B, seismic isolation remained technically feasible during concept design but was abandoned once budget, construction complexity and client acceptance became governing considerations. In both cases, identifying the dominant project constraints early would have reduced the need for subsequent redesign.

4.3. COMMUNICATING RISK AND PERFORMANCE TRADE OFFS

A recurring difficulty is that performance objectives are easy to state but their structural consequences are not obvious to those who set them. The expectation that a building remains fully operational after a major earthquake, as required for the building P in Project B, implies either an isolation interface or substantially stiffer and heavier members, implications that are seldom appreciated when the target is first agreed. When base isolation was rejected, the decision traded a clearly understood technical solution for a less visible increase in residual seismic risk and material quantity. The engineer's role is to make such trade-offs explicit in terms the client values, including capital cost, programme, maintenance and post-event downtime, so that the choice between elastic robustness, ductile economy and isolation is made knowingly rather than by default. However, to fully grasp the implications of certain decisions, especially for irregular and atypical buildings, a full analysis and design is usually required, which is rarely available at the time of decision making.

4.4. ITERATIVE DESIGN AND COMPROMISE

Neither project reached its solution in a single pass; both converged through iteration in which each discipline compromised something. In Project A the compromise was to accept higher reinforcement ratios under an elastic approach in order to preserve architecturally acceptable wall thicknesses and uniform detailing. In Project B it was to abandon the technically preferred isolation scheme for a conventional composite system whose heavier cores, walls and diagrid sections satisfied low to medium ductility while keeping the member sizes architecturally acceptable. These outcomes were not failures of design but the product of negotiated compromise, and in both cases the number of costly iterations would have been reduced by earlier alignment between architect, engineer and client on the constraints that ultimately governed.

5. ENGINEERING INSIGHTS & CONCLUSIONS

Comparing the two projects highlights a common discord between architectural ambition and seismic rationale, resolved differently according to the governing code, the building importance and the project constraints. The following themes draw the cases together.

5.1. ELASTIC VS DUCTILE SEISMIC DESIGN STRATEGIES

Ductile design dissipates seismic energy through controlled inelastic deformation, accepting damage in exchange for smaller forces and more slender members. Elastic design instead keeps the structure within its elastic range, trading economy for predictability and damage avoidance. Project A adopts elastic design to satisfy Italian practice, while Project B, although it adopts low to medium ductile design, results in large and costly structural elements due to high seismic and occupancy demands.

5.2. ROLE OF SYSTEM REGULARITY IN SEISMIC PERFORMANCE

Regularity in plan and elevation underpins the favourable, predictable inelastic behaviour that codes reward with higher behaviour factors. Project B departs from it: re-entrant plans, interrupted load paths and non-uniform stiffness depart from clean classification under EN 1998. Irregularity drives lower behaviour factors, more onerous analysis (modal response spectrum, time-history) and a reliance on elastic capacity rather than ductile mechanisms.

5.3. IMPACT OF CLIENT DECISIONS ON STRUCTURAL EFFICIENCY AND RISK

Client priorities decisively shape the structural outcome. In project A, the limited available arrangement of vertical elements coupled with unfavourable floor layout led to torsional flexibility which yielded significant reinforcement quantities. In Project B, concerns over cost, constructability and the unfamiliarity of seismic base isolation led to its rejection, despite clear technical merit.

Such decisions trade lower initial complexity and capital cost for greater material use and higher residual seismic risk – a transfer of risk from the construction phase to the building's in-service performance.

5.4. BALANCE BETWEEN CODE COMPLIANCE, ARCHITECTURE AND COST

Designing buildings in high seismic regions often requires structural engineers to balance competing project priorities. Clients may expect the building to remain operational after a major earthquake, while architects seek slender structural elements and flexible, unobstructed spaces. At the same time, project budgets place pressure on construction costs and material quantities. Improving seismic performance typically requires either a more robust structure, which can increase member sizes and costs, or the adoption of advanced technologies that may add complexity and specialist requirements.

The engineer's challenge is therefore to find a solution that achieves the required level of safety and performance while respecting the architectural vision and maintaining an acceptable project cost. Success depends on early collaboration between the client, architect, and engineer, together with a clear understanding of the trade-offs involved.

5.5. RECOMMENDATIONS FOR MANAGING ARCHITECT-ENGINEER CONFLICTS IN SEISMIC REGIONS

The cases suggest a small set of practical measures:

- Engage the seismic strategy at concept stage, before the geometry hardens.
- Translate performance targets such as Immediate Occupancy into tangible cost, member-size and programme implications the client can weigh.
- Identify and agree the governing constraint explicitly and revisit it only with full awareness of the cost of doing so.
- Assess alternative solutions, such as isolation, on technical merit and address perception and familiarity barriers directly rather than letting them decide by default.
- Sustain continuous architect–engineer–client dialogue to limit late, expensive iteration.

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