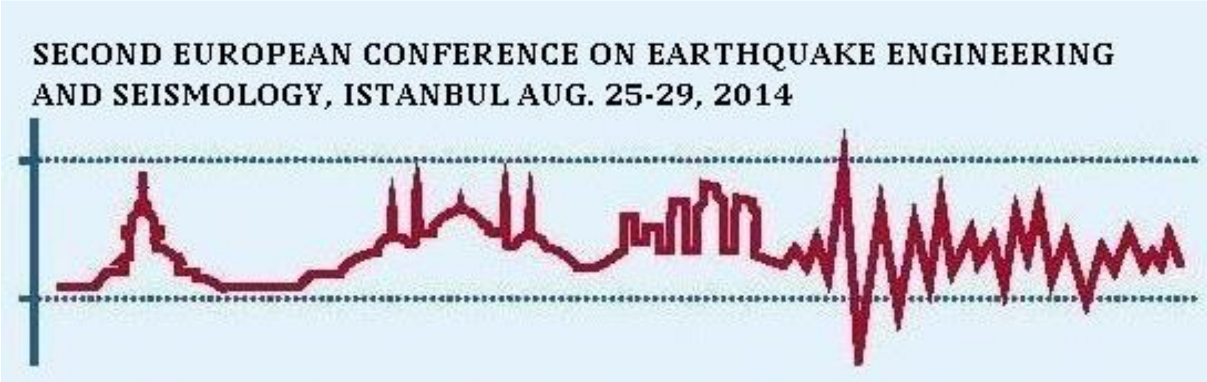

SEISMIC RISK MITIGATION FOR MISSION-CRITICAL FACILITIES: A CASE STUDY FROM PLANNING TO CONSTRUCTION OF A HIGH-PERFORMANCE DATA CENTRE IN COSTA RICA

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ABSTRACT

The consultants to a new high-end data centre in Costa Rica brought serious questions into focus at the concept design phase. The owner, a major financial institution based in San José, wanted the data centre of its new headquarters expansion to achieve Tier III certification. The Uptime Institute guidelines require Tier III facilities be designed to remain nearly operational during foreseeable events such as maintenance, loss of power and flooding. Should the owner expect this to include *downtime* due to damage by such infrequent, catastrophic events as major earthquakes? If so, how can the design team 'ensure' (*i.e.*, rationally design to meet) such performance? Are the needed tools and knowledge readily available?

A survey (Symantec 2010) among data centre owners revealed a *downtime* due to earthquakes of 9 hours per year on average. Post-earthquake reports by EERI and others show that the potential damage can take weeks to repair. Reports from recent earthquakes in Japan and the US showed how good design practices and the use of available technology can dramatically reduce damage to Data Centres so that they continue to operate immediately after major events. With this information at hand, the team quickly concluded that a standard office building structure could not ensure that the required performance would be on a par with that of the rest of the trades, but also concluded that the technology was readily available to design and build a structure to meet such performance objective.

This paper describes the entire process: from the early conceptual decision-making, based on sound earthquake engineering, through a formal site-specific and solution-specific seismic risk analysis, to the final design of an independent base-isolated structure. As part of the seismic risk analysis site-specific *displacement* spectra were developed, largely following the work of Faccioli *et al.* (2004, 2011) and data from the most recent hazard study for the region (Climent *et al*. 2008). The engineering tools ranged from conceptual order-of-magnitude estimates from first principles to state-of-the-art base isolation devices. The consultants framed the whole process within the concept of performance-based engineering using the direct displacement based design technique (Priestley *et al.* 2007) with a suit of non-linear response history analyses for design verification.

The result of the design process, which involved multiple disciplines as well as the owner, is an award winning, robust, high-performance and very-low-maintenance structure certified as Tier III by the Uptime Institute. The design meets the Costa Rica Seismic Code of 2010 but borrows many concepts from Eurocode 8 and ASCE 7-10.

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INTRODUCTION

Sometimes, the achievement of the mission and objectives of an organization depends on the reliability and availability of large databases. As such, these databases are *mission-critical components*, often stored and processed in sophisticated *data centres* of varying complexity and level of security. Their main common feature is their robust performance in terms of *uptime*, or time in operation. Many predictable events can hinder performance, resulting in *downtime*: maintenance, power shortages, ventilation systems failure among others. The industry referent *Uptime Institute* classifies the levels of performance of *data centres* into four tiers depending on their expected *uptime*, as shown in Table 1.

Table 1. Data Centre Tier Classification System and Typical Features

	Tier I	**Tier II**	**Tier III**	**Tier IV**
N. of main utility feeds	1	1	1 active + 1 passive	2 active
Redundancy of components	N	N+1	N+1	2(N+1)
Power consumption [W/m²]	200-300	400-500	1000-1500	>1500
Design live load [kN/m2]	425	500	750	>750
Unit cost[(1)] [US$/m2]	4,900	7,000	10,000	>12,000
Annual data *downtime* due to site [hrs]	28.8	22.0	1.6	0.4
Site uptime	99.671%	99.749%	99.982%	99.995%

Adapted from Turner & Brill (Industry Standard Tier Classification Define Site Infrastructure Performance).
Notes: [(1)] Unit cost C_I ~C_{IV} over effective area, A_{eff}

It is clear that natural phenomena such as earthquakes have the potential of causing downtime to data centres. This article postulates that earthquakes may lead to an annual average *downtime* in excess of the performance limits criteria for high-end *data centres* and this issue is not explicitly treated in common engineering practice during the planning phases of this type of projects. Many clients, even sophisticated ones, often assume that compliance with a seismic design code will automatically provide the expected level of performance. This paper shows that this may not be the case, formally presenting the issue from the point of view of earthquake risk and loss mitigation.

A recent survey by the IT security consultant Symantec (Symantec Disaster Recovery Study 2010) found that 46% of the surveyed *data centres* experienced *downtime* because of earthquakes in the preceding five years, with an average of 9.3 hours per year. This exceeds the maximum expected for a compliant Tier III data centre (1.6 hours, see Table 1). Consequently, an explicit evaluation of seismic risk is called for during the design of a high-tier data centre.

The discussion of this article centres on a case study of a recent project in which the authors were involved as lead structural designers from the early stages: the New Data Centre for Caja de ANDE in San José, Costa Rica, completed in February 2014. When this mutual funds manager did the program of requirements for an expansion of its office complex, the architects and clients planned a new Data Centre to be included in the ground level, above two parking basements. The rationale for this location was that: a) the utilities mains could easily feed from the basements; b) heavy equipment easily rolls-in from the street and the basement parking; c) two basements would prevent eventual flooding; and d) earthquake acceleration would be less than in upper storeys. All these are very good reasons and the planned location seemed straightforward.

Later into the design process, the client informed the design team that that the Data Centre was to meet Tier III requirements. The structural team noted that in the case of an earthquake even a well-designed *commercial office* building would require a significant *downtime* for repairs. The methodology described here predicted a mean annual *downtime* of nearly 8 hours. This is in the same general range as reported by Symantec's survey and much more than the 1.6 hours limit for a Tier III.

The ATC-58 methodology, the databases accompanying PACT Tool, and the implicit hazard curve from the Costa Rica Seismic Code (CFIA, 2010) give a good order-of-magnitude estimate of the mean annual *downtime*, E[D]. The building site is on ground class C per Eurocode classification (class D per ASCE-7) and the code-specified hazard for 10% probability of exceedance in 50 years (typically used for office buildings) in terms of PGA on rock is 0.3g.

OVERVIEW OF SEISMIC RISK OF CRITICAL FACILITIES

The present authors have often found that the owners of critical facilities provide detailed specifications to ensure the performance of most building systems but fail to do so for the structural system, and believe this to stem from a series of misconceptions. The philosophy of the design codes around the world is to save human lives and avoid excessive damage to the structures and contents. They rarely consider business interruption and losses in terms of downtime. As a result, critical facilities often reside inside commercial buildings, which will normally endure tolerable earthquake damage and consequently *downtime* for repairs. The authors believe that most owners still do not fully understand this issue and therefore pay less attention to the performance of the structure than to other systems.

At the other end of the spectrum, there are those who see the consequences of natural phenomena as unavoidable. Although it is true that they are random, we have the analytical tools to make rational assessments of such consequences. Here is where the concepts of loss assessment and risk mitigation come into play. A useful and generally agreed-upon framework for building loss assessment is that of ATC-58 (ATC, 2011) dubbed the PEER Performance Based Earthquake Engineering framework, which for the purposes of this paper takes the following form:

$$\lambda(D) = \iiint G\langle D|DL\rangle \, dG\langle DL|\theta\rangle \, dG\langle \theta|PGA\rangle \, d\lambda(PGA) \tag{1}$$

Where: $\lambda(x)$ is the probability of $X>x$ in a given time; G denotes a conditional probability of exceedance; D is the decision variable (*downtime*); DL is a measure of building damage (*e.g.* minor, moderate, severe); θ is an engineering demand parameter (inter-storey drift ratio); the peak ground acceleration PGA is the measure of ground motion intensity. The above is a mathematical representation of the risk convolution:

$$RISK = EXPOSURE * VULNERABILITY * HAZARD \tag{2}$$

The risk [$\lambda(D)$] is expressed here in terms of *downtime* (repair time) and the concept of *exposure* and *vulnerability* are merged together in the vulnerability function: $\{G\langle D|DL\rangle \, dG\langle DL|\theta\rangle \, dG\langle \theta|PGA\rangle\}$. This conditional approach is preferred because of the difficulty of computing $G\langle D|PGA\rangle$ directly. The process is broken down into three steps: the structural response analysis relates θ to PGA; the fragility functions relate θ with different levels of damage DL; and the consequence functions express the loss (in terms of *downtime* in this case) for each level of damage. All these steps use different methods and sources, which are typically of diverse origin. Only the first needs to be case-specific. The hazard expresses the probability of exceedance of the PGA at the site, usually represented by a hazard curve.

ESTIMATING DOWNTIME DUE TO STRUCTURAL REPAIRS

Downtime or time to repair is used here as the loss (or decision variable) within the PEER framework and risk is expressed as the expected mean annual value of *downtime*. Note that this paper uses two measures of time: t_r is the time (person-hours) to repair of a number of elements. D or *downtime* is the total time (calendar days) from the event to the restart of operations. When D or E[D] is a small number it is expressed in hours instead of calendar days.

Vulnerability

The fragility curves (Figure 1b) and consequence functions (Figure 1a) come from the PACT database. The latter are expressed as the time to repair each unit of work (*i.e.* one beam-column joint or one square wall panel). The average repair time decreases linearly with the number of elements to repair, as shown in Figure 1 (top left). There are n = 210 joints and n = 192 wall panels. Equation (3) gives the total *downtime* D for each group of elements shown in Figure 1a, where: $\overline{t_r}(n, DL)$ is the consequence function; N_{hpd} is the working hours per day (here 276); t_{mob} is the mobilisation time (here 7 days); and $t_{r|collapse}$ is the reconstruction time given that collapse occurs (assumed 180 days).

$$D(n, DL) = \frac{\overline{t_r}(n, DL)}{N_{hpd}} + t_{mob} \leq t_{r|collapse} \tag{3}$$

The fragility functions are combined with the consequence functions to produce the vulnerability functions of Figure 1c through the following relationship: $D(\theta) = \sum D(DL_i) \cdot P(DL_i|\theta)$, where $P(DL_i|\theta) = G(DL_{i+1}|\theta) - G(DL_i|\theta)$. Note that the latter are structural fragilities already discussed.

Structural Response Analysis

It is necessary to know the response of the structure in order to relate θ to PGA. A pseudo-incremental dynamic analysis response curve can be estimated (Figure 1d) through displacement-based design (Priestley, Calvi, & Kowalsky, 2007) and the displacement spectrum. The building is 32x60m in plan and 6 storeys high of reinforced concrete walls and frames. T_1 is taken as 1.2s and the total mass 12700 tonne. The elastic spectrum in the base code has T_B=0.6s, T_C=3.91s. Table 2 shows the basis of design.

Table 2. Basis of design for office building expansion

Design basis code	Costa Rica Seismic Code (CFIA, 2010)
Location	San José, Costa Rica
Ground condition	Stiff soil with $180 < V_{S30} < 360$m/s
Occupancy	Commercial
Importance	Ordinary ($\gamma_I = I = 1.0$)
Lateral force resisting system	Dual, high ductility ($\mu_\Delta = 4$)
Performance objective	Life safety (*eg*, ultimate strain in steel $\varepsilon_u = 0.06$) for 10%/50yr (PGA=0.36g)

The pseudo-IDA curve is obtained from a pushover curve. In this case, a bilinear approximation to the pushover curve was obtained by estimating Δ_y, Δ_d, $V_{b,y}$ and $V_{b,d}$ based on DDBD principles. The values of PGA for each θ are found from

$$S_a(T,\mu) = PGA \cdot\ 2.5 \cdot R_\mu \cdot \frac{T_B}{T} \tag{4}$$

Equation (4) is solved for PGA with $T = T_e = 2\pi\sqrt{m_e\Delta/V_b(\Delta)}$, $\mu = \Delta/\Delta_y$. $V_b(\Delta)$ is simply the pushover curve. The interstorey drift θ can be obtained from the pushover analysis for each value of Δ or can be estimated as $\theta_y + \theta_p\cdot(\mu-1)$. R_μ is obtained directly from μ as described in EC8 (CEN, 2004). T_B = 0.6s for the CRSC-2010 elastic spectrum applicable to the site.

Note that this procedure does not give us any information regarding the dispersion. Therefore, one must either work deterministically with mean values and bear in mind this limitation or assume reasonable values for the dispersion following the guidance of ATC-58. Another simplification worth noting is that θ_y corresponds to the ground floor because the repair works on upper floors do not delay the re-opening of the critical facility of interest. Were it not for this simplifying assumption, each floor would require a separate analysis because the damage will likely vary throughout the height of the building.

Hazard

The hazard curve used here is an approximation, but is consistent with the requirements of the Costa Rica Seismic Code. At the building site (180m/s < V_{S30} < 360m/s) the PGA for 10% probability of exceedance in 50 years (475-year event) is 0.36g. Important buildings must be designed with importance factors of 1.25 (2%/50yrs or 970-year event) and 1.5 (5%/75years or 1462-year event). The annual probabilities of exceedance of these events are 0.21%, 0.10% and 0.07% respectively. The relationship found in EC-8 (CEN, 2004) $\lambda(PGA) = k_0\, PGA^{-k}$ gives the hazard curve of Figure 1f , where k_0 and k were estimated through linear regression analysis.

A note on uncertainty

There is significant uncertainty in the results shown, much of which this paper does not quantify. A rigorous analysis wants the dispersion in both the consequences and the capacity accounted for, a task best done through stochastic simulations. PACT provides the tools for this. Therefore, although this paper follows a rational approach, this is far from the state-of-the art in loss analysis in terms of *downtime*. Here we shall simply note that: a) the estimated *downtime* is so much beyond the limit that it is best to spend efforts in mitigation; and b) these results are most useful in comparative terms.

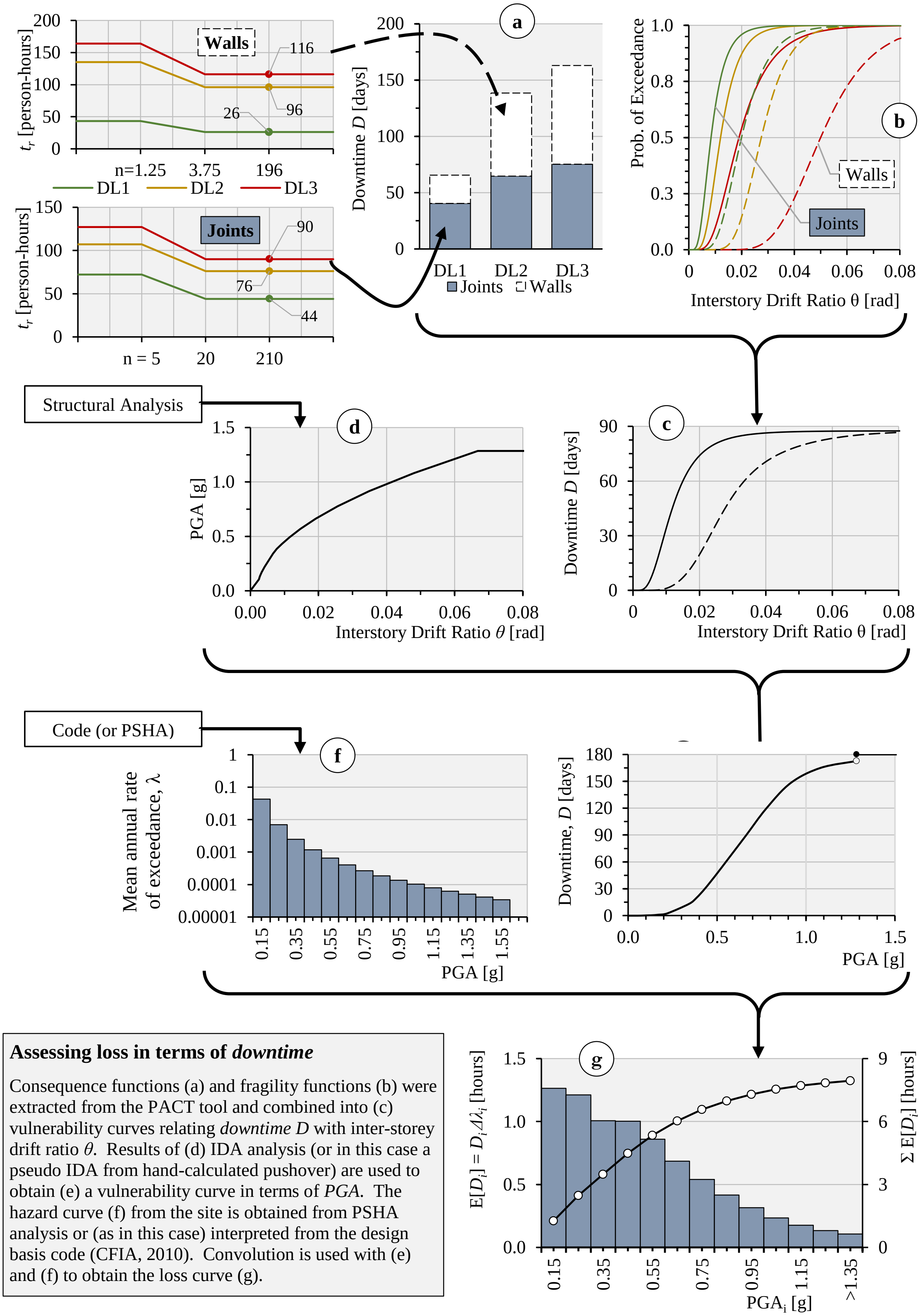

Figure 1. Risk analysis flowchart showing results for the case study. The area under the loss curve gives the expected annual loss in terms of *downtime*, which in this case amounts to nearly 8 hours.

Actually, the authors show elsewhere (González & Zamora, 2013) that much simpler calculations would suffice as an argument against placing a Tier III facility within a commercial building in a high-seismicity region. For example, if we assume just two events with mean annual rates of exceedance $\lambda_1 = 1/_{475}$ and $\lambda_2 = 1/_{1500}$ respectively with *PGA*=0.36g and 0.54g causing moderate damage with D_1 = 30-days *downtime* and complete damage with D_2 = 180-days *downtime*, we can make a simplified assessment, which is still beyond the 1.6 hours limit:

$$\mathrm{E}[D] = \Sigma\Delta\lambda_i \cdot D_i = \left\{\left(\frac{1}{475} - \frac{1}{1500}\right) \cdot 30 + \left(\frac{1}{1500}\right) \cdot 180\right\} \cdot 24 = 3.9 \text{ hours}$$

OPTIONS FOR RISK MITIGATION

Despite the above results and the findings of the Symantec survey, Data Centres can be designed to remain operational after major earthquakes. When the design team explicitly addresses seismic risk, mitigation strategies can be subsequently implemented with very satisfactory results. Examples of such good performance are those of Data Centres in Japan after the 2011 Earthquake and tsunami, and in the US West Coast after an event in that same year. No *downtime* was reported in either of these cases.

Risk can be mitigated once it is understood and, for this purpose, the causal relations leading to *downtime* must be broken down. We can classify these first by their scale: *building, local, regional* and *global*. Design decisions have the capacity to influence most the small-scale effects, while strategic planning is most effective in dealing with large-scale effects. We must then decide which factors of risk can be mitigated most effectively: *exposure, vulnerability* or *hazard*. Effective mitigation requires the reduction of at least one of these factors.

Exposure is the value of the assets at risk (*i.e.* liable to suffer damage). The value of the data stored in the data centre is not relevant in the estimation of *downtime* or it can be said to be irreplaceable and thus inestimable. However, if the data stored is only one of several *copies* available in *redundant* sites, the exposure is now null and the *downtime* can be zero regardless of the other factors. This is true as long as the probability of damage to all the data centres due to the same event is insignificant (in other words, if the sites are far enough from each other).

Hazard is the probability of a given intensity of ground motion being exceeded in a certain time at the site. It can be reduced through suitable site selection. For example, away from active faults, on stable non-liquefiable ground and beyond the maximum tsunami wave penetration. When the site is given, as in our case study, the hazard cannot be mitigated. As in the case of the exposure, distant redundant sites removes the hazard.

Vulnerability is the damage or loss caused by a ground motion of a given intensity. It does not depend exclusively on the structure but also on its components: a chain only as strong as its weakest link. Reduction of physical vulnerability requires consideration of access routes, mains feeds, campus, the structural system, support systems, raised flooring, equipment racks, the distribution lines and suspended ceilings. The strength of the systems can be increased or the building can be base-isolated. In this example several mitigation measures are applied simultaneously as will be explained.

It is necessary to determine whether the current risk is acceptable or not, and then which would be the acceptable level of risk that needs to be attained. A common way to visualize risk is through a *risk matrix* in which *performance objectives* can be readily expressed as shown in Figure 2 (left) based on SEAOC Vision 2000 document.

An annual *downtime* E[*D*] in the order of 1 to 10 hours is acceptable for Tier I and Tier II data centres (refer to Table 1) but unacceptable for a Tier III or IV which require not to exceed 1.6 and 0.4 hours/year respectively. A better performance is required, which implies moving towards the bottom left corner of the matrix. The exercise of Figure 1 is repeated assuming this time that the building is designed for a less-frequent event (970 years instead of 475 years) and for a better performance (in this case decreasing the steel rebar strain from 0.06 to 0.03 to reduce the crack size –this relates directly to less repair time). The resulting risk curve, smaller by one order of magnitude, is shown in Figure 2 (right). E[*D*] is 0.65 hours/year.

In other words, a stronger structure can achieve the required performance –as would be expected. This improvement comes at a significant cost premium through larger design base shear, which goes from 0.24 to 0.69 times the weight –a threefold increase. The cost increase is equivalent to 40% of

$A_{eff} \cdot C_{III}$ (see Table 1), clearly not an attractive option. It would not make sense to strengthen just a few bays or storeys. The conclusion was to build the data centre detached from the main office building expansion and design for a higher performance. A decision with such impact in the overall project would have been less obvious without the rational background provided by the risk analysis shown.

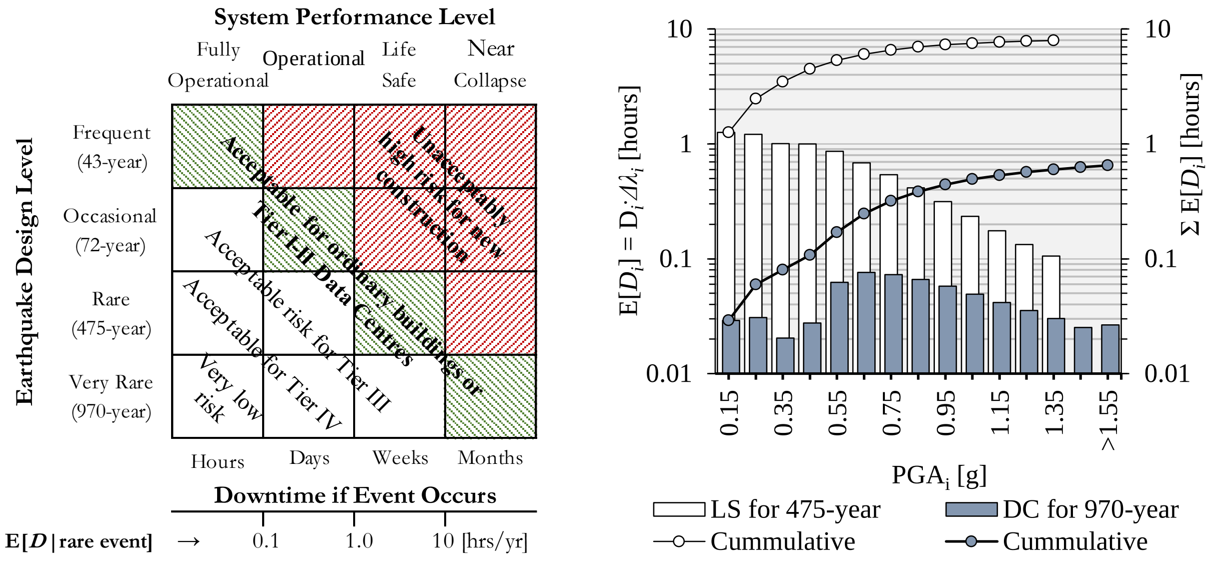

Figure 2. Left: Risk matrix (by the authors, based on Vision 2000). Right: Loss curve (binned and cumulative) for the ordinary (white) and the improved (shaded) designs.

A complete seismic risk mitigation must take into account all the possible sources of risk –direct and indirect. Table 3 breaks down the risk into different scales, from building to global scale, and briefly describes some of the mitigation measures applicable to each. Note that providing redundancy is one of the most effective mitigation measures. Those measures implemented in the case study are marked in the table. Although the team discussed the option of building a mirror data centre, the client did not considered it economically viable in this particular case.

Table 3. Breakdown of seismic hazards and corresponding available mitigation measures

Scale of Effects	Risks	Mitigation Measures	Stage (1)
I. Building	I.1 Non-structural damage:	I.1a. Brace raised floor (2)	D
	i. Equipment	I.1b. Anchor racks to floor slab (2)	D
	ii. Information systems	I.1c. Use seismic rated suspended ceiling	D
	iii. Backup systems	I.1d. Laterally brace pipes, conduit and cable trays (2)	D
	iv. Other non-structural	I.1e. Reduce inter-storey drift through stiffer structure (*eg* walls, braces) (2)	C
	damage	I.1f. Make systems redundant (2)	P,C
	I.2 Structural damage:	I.2a. Design for a less frequent event (eg 970-year instead of 475-year) (2)	D
	i. Damage requiring repair	I.2b. Design for higher performance (eg 'damage control' instead of 'life	D
	ii. Building (near) collapse	safe') (2)	
	iii. Unrepairable damage	I.2c. Make structure regular and redundant(2)	C
		I.2d. Isolate facility (or entire building) from ground motion(2)	P
II. Site	II.1 Disruption of mains feeds	II.1a. Make feeds redundant(2)	P
		II.1b. Provide seismically-designed supports(2)	D
	II.2 Soil liquefaction	II.2a. Select site sensibly	P
		II.2b. Improve ground at site	C
		II.2c. Use special foundation systems	C
III. Regional	III.1 Disruption of lifelines	III.a. Provide redundant lifelines (2)	P
		III.b. Make data centre redundant (mirrored or multiple sites)	P
	III.2 Landslides	III.c. Select site sensibly	P
		III.d. Stabilize slopes through geotechnical engineering	C
IV. Global	IV.1 Tsunamis	IV.a. Make data centre redundant (mirrored or multiple sites)	P
		IV.b. Select site sensibly	P

Notes: (1) As a general reference, consider three project stages: (P) planning, (C) conceptual design and (D) detail design.
(2) This project implements these mitigation measures.

The final decision to make was whether to seismically isolate the detached structure. A displacement-based preliminary design followed by a risk analysis as in Figure 1 of the isolated and non-isolated structures, yields E[D] = 0.28 and 0.89 hours/year, respectively. In other words, base isolation reduces risk by nearly 70%. The cost premium is 14% and 9% of $A_{eff} \cdot C_{III}$ respectively. Even with the slightly greater cost, the team decided to incorporate base isolation technology.

DESIGN OF THE DATA CENTRE

Based on the considerations explained above, in the final concept the data centre is an essential facility, which must remain operational after a rare earthquake and not requiring lengthy repairs. It is a detached building, removed from the main office building so that the design of the latter needs not be affected by the new, increased design criteria. Furthermore, since damage to the main building would affect any utilities going through it as well as access and egress to and from the data centre, the detached building also incorporates all essential utilities and those serving both buildings. These included all the emergency and backup power systems. Finally, the design team decided to use a base isolation system that would allow a more rational control of the damage by relocating the damageable parts to areas easily accessible without interfering with the operation of the data centre.

Design Criteria

The facility is a detached reinforced concrete structure with shear walls in both orthogonal directions. The layout of resisting elements is very regular and symmetric in order to make the structural response as simple and predictable as possible. The structure is isolated through nine identical double-curvature friction pendulums, symmetrically distributed over concrete pedestals on spread footings coupled by deep grade beams. A 500mm-thick concrete slab rests on the pendulums and provides sufficient stiffness and development length for the support of the walls above. The two-storey high superstructure is also symmetrical although the architectural layout is not, which means that some partitions are released from the lateral system in order to maintain structural symmetry. Displacement-based design is the framework of choice for sizing the isolation system, as it is more straightforward for this purpose than a force-based approach.

Seismic Hazard for Design

The Costa Rica Seismic Code (CFIA, 2010) prescribes for the site (in the city of San José) an effective PGA of 0.36g (Zone III, soil profile S3). For an essential facility, an importance factor of 1.25 is required so the design PGA is 0.45g. Figure 3 shows the acceleration and displacement spectra used for design.

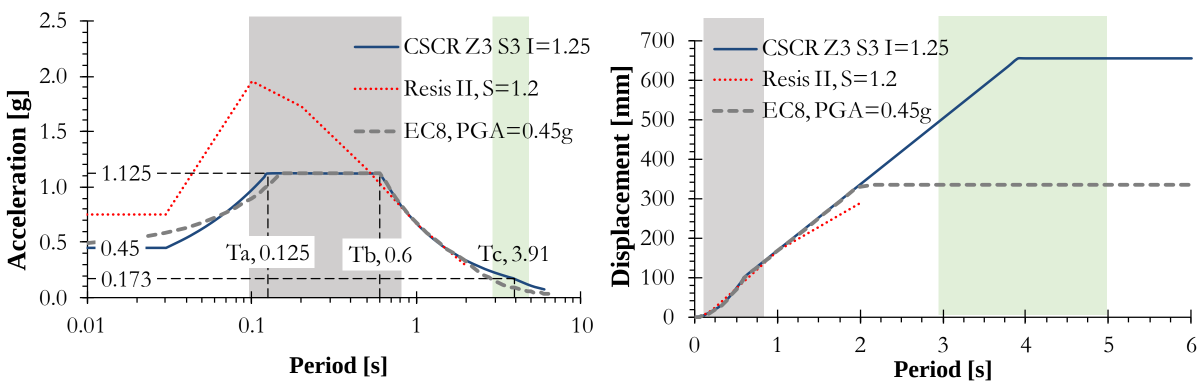

Figure 3. Left: acceleration spectra for firm soil and 970-year return period from three sources (CFIA, 2010; CEN, 2004; Climent, Rojas, Alvarado, & Benito, 2008). Shown shaded in gray is the range of interest for rigid structures 1-2 storeys high. Right: corresponding displacement spectra. The green-shaded area is the range of interest for isolated structures as the one under study.

DESIGN OF THE ISOLATORS

Arguably, the optimal design for the isolation system would consist of four devices symmetrically distributed. In such way, each would get approximately the same gravity load. This would be an attractive feature because the axial load on the isolators determines their post-yield stiffness. However, the footprint of the building is rather large (11x12m) and such layout would require a very indirect load path for gravity loads and potentially long-term deflections and cracks under permanent loads. A layout with 12 units provides the most direct load path. The compromise layout adopted uses nine devices (3x3 grid).

The gravity load on the base isolation system is important because its apparent stiffness depends on the vertical load. The distribution of loads is of less importance because the displacement of each unit is the same, the system is approximately symmetric and the increase in stiffness in one element follows a corresponding reduction in another. Thus, the average load is more important for the design. Figure 4 shows a schematic of the exact double-curvature friction pendulum used, next to its hysteretic response curve. The total gravity load including the live load acting concurrently with the seismic loads is 7460kN.

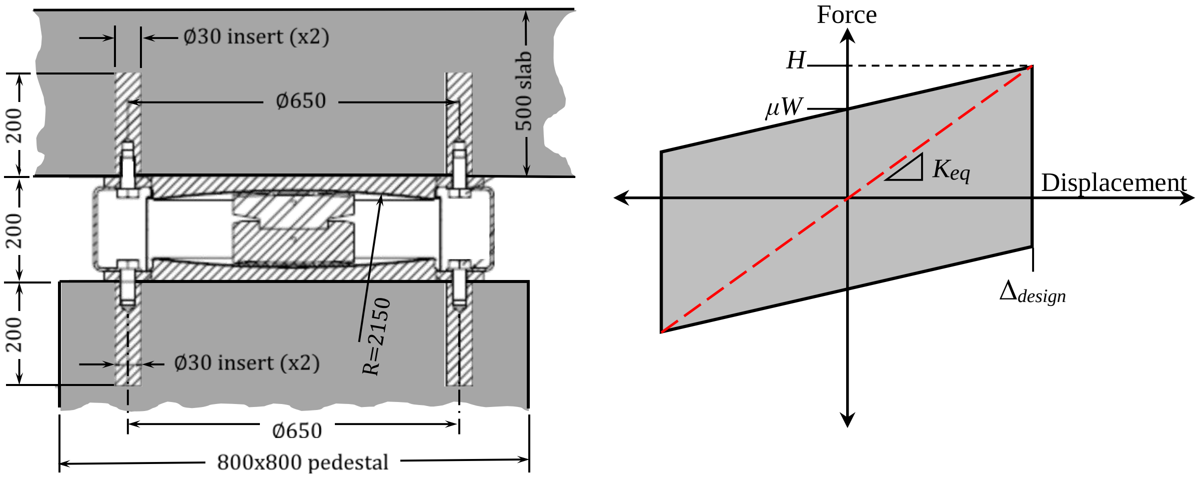

Figure 4. Schematic of a double-curvature friction pendulum (left) and its characteristic hysteretic curve (right). Source: Alga ® project drawings.

Design Parameters

The radius of the concave surfaces R and the dynamic friction coefficient μ_{dyn} are the two factors defining the response of a friction pendulum. In double-curvature pendulums, an equivalent radius $R_{eq}=2\cdot(R-h)$ is used instead, where h is the distance between the surfaces. An interactive process was followed leading to the selection of pendulums with R_{eq} =4000mm and μ_{dyn}=3%. Equations (5) through (7) give the effective stiffness, effective period and maximum horizontal force in each pendulum.

$$K_{ef,i} = \frac{W_i}{R_{eq}} - \frac{\mu_{din} W_i}{\Delta} \tag{5}$$

$$T_{ef} = 2\pi \sqrt{\frac{W_i}{K_{ef,i} \cdot g}} \tag{6}$$

$$V_{max} = K_{ef} \cdot \Delta \tag{7}$$

Here, W is the axial (gravity or gravity plus seismic) load and Δ is the system displacement. Note that the equivalent stiffness and period depend on the displacement, as is the case in any non-linear system. The equivalent system stiffness K_{ef} is obtained through adding the equivalent stiffness of each pendulum or, equivalently, substituting W_i with $W = \sum_{i=1}^{N} W_i$ for all N devices. Note that the displacement is the same for all the pendulums, as are the radius and friction coefficient. Further note that the ratio W_i /$K_{ef,i}$ is constant for constant values of R_{eq}, μ_{din} and Δ so the equivalent period of the system, T_{eq}, is the same as the period of any pendulum; alternatively the effective period of the system can be obtained from equation (6) with W and K_{ef}. Therefore, we can temporarily ignore the distribution of gravity forces for the selection of the isolators. Nevertheless, two facts must be borne in mind: the most loaded device must be able to carry the imposed axial load and the axial force in the least loaded device must never become zero.

Selection of the Devices

The geometric parameters of the devices (R_{eq} =4000mm and μ_{dyn}=3%) were selected through the iterative process described next. The system is idealised as an equivalent single-degree of freedom system. The spectra used so far to describe the seismic hazard are elastic spectra for 5% damping. The energy dissipation through friction is analogous to additional damping, often called *hysteretic* damping, which we must quantify. We need an initial estimate so we assume ξ_{hyst}=20%. The spectral ordinates are reduced, as recommended by Priestley *et al*. (Priestley, Calvi, & Kowalsky, 2007), through:

$$R_\xi = \sqrt{\frac{0.07}{0.02+\xi}} \tag{8}$$

Where $\xi = \xi_{el} + \xi_{hyst} = 0.25$, giving $R_\xi = 0.51$. From the reduced spectrum for 25% damping the maximum spectral displacement is 344mm for $T > T_C$=3.91s. We must now assume a performance point on the reduce spectrum. A good initial point is the corner period and displacement $\{T^{(0)},\Delta^{(0)}\}$={3.91s,344mm}. Equations (5) and (6) provide the equivalent period of the friction pendulum system, which for 344mm displacement is:

$$K_{ef} = \frac{7460\text{kN}}{4000\text{mm}} - \frac{0.03 \cdot 7460\text{kN}}{344\text{mm}} = 2516\text{kNm}$$

$$T_{ef} = 2\pi\sqrt{\frac{7460\text{kN}}{2516\text{kNm}\cdot 9.81\,{}^{\text{m}}/_{\text{s}^2}}} = 3.45\text{s}$$

Note that 3.45s is less than the period initially assumed. In addition, we must reassess the hysteretic damping. We use the ratio of areas enclosed by the elastic and hysteresis curves:

$$\xi_{hyst} = \frac{2}{\pi}\left(\frac{\mu_{dyn}}{\mu_{dyn} + {}^{\Delta}/_{R_{eq}}}\right) \tag{9}$$

For $\Delta^{(0)}$=344mm we get $\xi_{hyst}^{(1)} = 16.5\%$ and consequently a new $R_\xi = 0.55$. After a few iterations, the results are as shown in Table 4. Note that V_{base} comes from Equation (7).

Table 4. Final design parameters.

Parameter	**Result**
T_{design}=T_{ef}	3.38s
Δ_{design}	297mm
K_{ef} (system)	2.63MNm
ξ_{hyst}	18.4%
R_ξ	0.52
V_{base}	781Kn

The design displacement is then Δ_{design}=300mm. The selected isolator is Algasism® model APS2500/600 manufactured by Alga S.p.A. with low-friction Hotplate® surfaces, which maintain the low friction at high temperature. Geometric specifications: ∅ = 600mm, R_{eq} = 4000mm, μ_{dyn} = 3%, N_{max} = 2500kN.

Distribution of Gravity Loads

It is necessary to verify that the most loaded device will not exceed its maximum capacity and that the least loaded will not lift up. The mass is approximately distributed in three equal parts in the roof, floor and base slab so the overturning moment becomes: $OTM = \frac{1}{3}V_{base}\sum h_i = \frac{1}{3}781\text{kN}\cdot(8.3\text{m}+3.9\text{m}+0.3\text{m}) = 3250\text{kNm}$. We can conservatively assume that only the supports directly beneath the walls resist overturning, so the vertical forces would be as shown in Table 5. The design is adequate because no device exceeds 2500kN or lifts up.

Table 5. Distribution of axial forces on the isolators.

Position	Gravity [kN]	Axial Force Seismic [kN]	Total [kN]
Corner (4 units)	569	±335	904 / 234
Intermediate (4 units)	922	±670	1593 / 252
Central (1 unit)	1495	±670	2165 / 825

DESIGN VERIFICATION

We use non-linear response history analysis to verify the adequacy of the design. One single non-linear link element with a bilinear curve models the entire set of isolators and a single point mass represents the superstructure. Its yield force is $\mu_{dyn}\cdot W$ = 0.03·7460kN=223.8kN, second stiffness W/R_{eq}=7460/4000=1.86MN/m, stiffness ratio $1\cdot10^{-5}$. The mass of the system is 760ton.

Climent *et al*. (2008) show that the hazard in the City of San José is dominated by cortical events with M_w = 6.75±0.25 and r = 15±5km. A set of seven unscaled "scenario" records matching these parameters give a median maximum displacement $\bar{\Delta}_{max} = 94\text{mm}$, probably too low to represent the hazard at the site. These records have a mean PGA of 0.27g. The same set scaled by 0.45g/PGA gives $\bar{\Delta}_{max} = 170\text{mm}$, comfortably below the displacement capacity of the isolators, and a maximum base shear V_{base} = 540kN. The residual drift is 5mm. The design appears to be satisfactory.

The design code has a very long corner period (T_C = 3.91s), which leads to large displacement spectral ordinates in the period range of interest. The preceding analysis seems to indicate that such large displacement demands are not likely to be present at the site. Nevertheless, it is possible to find a set of seven spectrum-compatible scaled records compatible with the seismic scenario of interest, through the PEER tool (Silva, n.d.). This set of records gives $\bar{\Delta}_{max} = 297\text{mm}$, median residual drift 11mm and median V_{base} = 775kN. Note the close match with the estimate: 297mm and 780kN.

CONCLUSIONS

Data centres provide a critical service to their users and it is essential that they remain operational most of the time. A major earthquake can potentially cause unacceptably long *downtime* to a data centre, even considering the infrequency of the event. Earthquake engineering and particularly performance-based engineering and risk analysis provide the set of tools necessary for a rational approach of this issue. The concepts and technology are available for structural engineers to mitigate this risk. The data centre designer, the client and the structural consultant – as a team – should explicitly address the design decisions needed to achieve the required performance objectives.

The Uptime Institute has certified the new data centre of Caja de Ande, completed in February 2014 in San José, Costa Rica, as Tier III. The design team used state-of-the art earthquake engineering techniques, including the explicit consideration of the risk for *downtime* and the use of base isolation technology. To the authors' knowledge, it is the first base-isolated building in Costa Rica.

Installing the data centre within a new office building –as originally planned– would encompass a mean annual *downtime* of nearly 8 hours, much greater than the 1.6-hour limit for a Tier III. The analysis done by the authors led to installing the data centre in a detached, base-isolated building. This permitted reducing the mean annual *downtime* to 0.28 hours. The structure was fitted with nine friction pendulums with 300mm maximum displacement, 4000mm equivalent radius and 3% dynamic friction coefficient.

Non-linear time history analyses with 14 spectrum-compatible records result in a median peak displacement of 300mm. The median displacement response to non-linear time history analysis with another 7 scenario ground motions compatible with the hazard disaggregation (M_W=6.75, r=15km) scaled to PGA=0.45g is 170mm. The corresponding base shear is 540kN.

The base shear on the superstructure at the over-strength of the isolators is 976kN or 0.13·W for a 970-year event. A conventional (bunker-like) design would require a base shear of 0.41·W. The base-isolated structure is more expensive by about 5%.

PROJECT CREDITS

Project:	New Data Centre Avenida Segunda, San José, Costa Rica
Owner:	Caja de Ande
Architects:	Adrián Guzmán, Guillermo Rojas (APlus Arquitectura SA)
Structural Engineers:	Alfredo González MSc, Leidy Zamora, Eduardo Guevara MEng, Felipe Calcáneo (FSA Ingeniería & Arquitectura SA)
Data Centre Consultant:	Data Center Consultores SA
General Contractor:	Compañía Constructora Volio & Trejos SA
Seismic Devices Supplier:	Alga SpA

REFERENCES

ATC (2011). *Seismic Performance Assessment of Buildings (ATC-58-1).* Applied Technology Council.

CEN (2004). *EN1998-1:1994 Eurocode 8: Design of structures for earthquake resistance. Part 1: General rules, seismic actions and rules for buildings.*

CFIA (2010). *Código Sísmico de Costa Rica.*

Climent Á, Rojas W, Alvarado GE, & Benito B (2008). *Proyecto Resis II: Evaluación de la amenaza sísmica en Costa Rica.* NORSAR.

González A, & Zamora L (2013). Riesgo sísmico y continuidad de operaciones: Diseño y construcción de un centro de datos con aislamiento de base en Costa Rica. *Estructuras 2013.* San José, Costa Rica: ACIES.

Priestley M, Calvi G, & Kowalsky M. (2007). *Displacement-Based Seismic Design of Structures.*

Silva W. (n.d.). PEER Ground Motion Database. Retrieved from http://peer.berkeley.edu/smcat/search.html

Symantec Disaster Recovery Study 2010. (n.d.).

Turner WP, & Brill KG (n.d.). Industry Standard Tier Classification Define Site Infrastructure Performance. *Site Infrastructure White Paper*. The Uptime Institute.